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NONCONNAH CREEK SEDIMENTATION STUDY

ANALYSIS USING A NUMERICAL MODELING APPROACH

by

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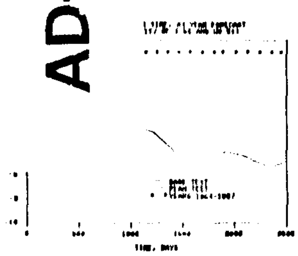

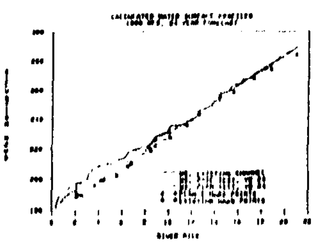

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This is a sedimentation analysis of a local flood protection project extending for about 20 miles along Nonconnah Creek, Memphis, TN. It was performed using the US Army Engineer Waterways Experiment Station version of the numerical model, HEC-6. This investigation is considered to be an analysis rather than a numerical sedimentation model study because Nonconnah Creek has been too highly disturbed over the past several decades to permit model confirmation. The justification for using the HEC-6 computer program as an analytical tool stems from its treatment of movable boundary hydraulics processes and its capability to analyze the project reach, the approach channel, and the exit channel as a stream system. The results show the project reach to be more stable than that reach would be without the project; however, the project channel invert shows a degradation trend. Both the design flood hydrograph and a 30-year period of record synthesized from rainfall were used in the analysis.

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PREFACE

This sediment study was conducted jointly by the US Army Engineer Waterways Experiment Station (WES) Vicksburg, MS, and the US Army Engineer District,Memphis, Memphis, TN (LMM), at the request of LMM.

It was conducted during the period May-September 1988. Mr. Frank Herrmann, Jr., was Chief of the Hydraulics Laboratory, WES, and Mr. Marden B. Boyd was Chief of the Waterways Division. Mr. William A. Thomas, Hydraulic Engineer, Waterways Division, designed and guided the study. Mr. Jerry Webb directed the work in LMM, performed much of the data collection, and made the computer runs. He was assisted by Mr. David Berretta of the Hydraulics Branch, LMM. Mr. Dewey Jones was Chief of Hydraulics Branch, LMM, during the study.

Commander and Director of WES during preparation of this report was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin. Commander of LMM was COL O'Brene Richardson.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
Fahrenheit degrees	5/9	Celsius degrees or Kelvins*
feet	0.3048	metres
miles (US statute)	1.609347	kilometres
cubic yards	0.7645549	cubic metres
foot-pounds per second per square foot	14.600	watts per square metre
tons (2,000 pounds, mass)	907.1847	kilograms

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F-32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F-32) + 273.15$.

NONCONNAH CREEK SEDIMENTATION STUDY

Analysis Using a Numerical Modeling Approach

PART I: INTRODUCTION

Description of Project

1. Nonconnah Creek is located in Memphis, Tennessee, Figure 1. It is tributary to the Mississippi River via McKellar Lake. The drainage area is 183.1 square miles, and topography varies from gently rolling hills and ridges in the upland area to a moderately wide valley as the creek approaches the Mississippi River flood plain. The stream slope is approximately 5 ft per mile. The existing channel is rather narrow and deep in the project reach; it will contain the 100 year flood with very little overbank flow.

2. The channel and flood plain have been used for borrow material over the past 15 years, and the flood plain has been encroached by fill areas. In 1986 the basin was 43 percent urban, and projected land use trends indicates it will be 66 percent urban by 2043. Within the past decade one bridge failed and several others had to be protected with riprap or gabions.

3. The Corps of Engineers is formulating a local flood protection project between River Miles 0 and 18.20. The plan tested in this study provides for vegetative clearing from RM 0 to RM 2.18. A 100 ft bottom width trapezoidal channel will be constructed between RM 2.18 and 7.9. Only vegetative clearing is proposed between RM 7.9 and 8.49. A 100 ft bottom width trapezoidal channel is proposed from there to RM 10.46. From RM 10.46 to 18.2 vegetative clearing is proposed. The channel is designed to contain the 100-year flood with almost no overbank flow which is similar to the existing channel. Because that design does not decrease the stress on the channel bed, numerous "hard points" have been added to stabilize the invert elevation. Therefore, bridges and pipeline crossings will be protected against channel bed erosion. The Corps of Engineers believes the proposed project, called Plan 1 in this report, will be more stable than the existing channel will be over the next 25 years; however, the design does not completely arrest aggradation or degradation. The purpose of this study is to provide the technical basis for informing the project sponsor about the long term stability of the invert of the project channel.

Approach

4. The WES computer program, "Sedimentation in Stream Networks" (TABS-1) was used to investigate the stability of the proposed channel design by forecasting channel aggradation and degradation over the next 10 to 25 years. Nonconnah Creek has been disturbed too severly to permit the normal model confirmation. Therefore, this investigation was a study in which a numerical model was used, but the procedures were less rigorous than numerical modeling because adequate field data to confirm the model could not be obtained from this highly disturbed study area. Although the procedures do not meet standards to be called a numerical modeling study, the study combined engineering judgement with a theoretical treatment of the degradation/aggradation processes along Nonconnah Creek. It utilized the fullest extent of present mobile boundary technology to study a project in which mobile bed hydraulics, and the channel bed dynamics associated with such fluvial processes, are expected to be highly significant during the life of the project. The objective is to predict the probable aggradation and degradation of the stream bed profile as the creek responds to future hydrology and future sediment discharges. The test for improvement was made by comparing the calculated aggradation and degradation with the proposed project to that of the existing channel assuming no project were constructed. Rather than instantaneous rates, the computer allowed the long term behavior to be evaluated. Moreover, the numerical model developed for this study provides the framework for a numerical model which could be confirmed and used in monitoring the behavior of Nonconnah Creek provided adequate field data were collected.

5. This study follows the numerical modeling approach in which a Base Test is run followed by the project Plan Test, but a rigorous model verification/confirmation was not included, as discussed below. The results of these two tests are compared to predict how well the proposed project will perform. In this study, the Base Test contains the existing channel geometry, n values, contraction and expansion coefficients, inflowing sediment discharge, inflowing sediment sizes, and bed sediment gradation. It contains the future water discharge hydrographs and a tailwater rating curve. The Plan Test is that same data set except where the project will modify channel cross sections and n values or will add hard points. The base test is described in Part VI, "Base Test." The performance of the proposed project is discussed

in Part V, "Project Conditions."

6. In preparing to make these computer forecasts, an inventory of available data was made and an analysis of the historical behavior of Nonconnah Creek was made covering the past 35 years. The conclusions and recommendations at the end of this report are based on this historical analysis as well as the computer forecasts.

Part II: INVENTORY OF PROTOTYPE DATA

Project Area and Study Area Boundaries

7. The project reach starts at the mouth, River Mile (RM) 0, of the creek and ends at RM 18.20, Figure 1. The study reach starts at RM 0 and extends to RM 21.005 to allow the impact of the project on the approaching channel to develop. The study area extends to the limits of flooding on either side of the channel. Six tributaries enter Nonconnah Creek along this reach, Table 1.

TABLE 1. Tributary Inflow Points

Name	River Mile	Drainage Area, Sq Mi
Days Creek	6.16	10.01
Hurricane Creek	7.80	7.70
Black Bayou	9.46	7.03
Ten Mile Creek	9.46	9.03
Johns Creek	11.94	26.08
Howard Road Outfall	14.86	3.24

Aerial Photographs

8. Four aerial mosaics were located for RM 0 to RM 21, and two earlier overflights were located for RM 0 to RM 3.

Streamgage Records

9. There is a USGS gage on the Nonconnah Creek at River Mile 17.34, the Winchester Bridge. The drainage area is 68.2 square miles at this site. Intermittent data has been collected on tributaries at irregular intervals.

Hydraulic Data

10. No high water profiles are available for existing channel conditions.

Suspended Sediment Data

11. No suspended sediment measurements are available.

Bed Gradation Data

Samples from the Existing (1988) Channel Bed

12. In April 1988 Memphis District personnel collected fifty-four sediment samples from the channel bed at 27 locations spaced along the 20 mile study reach. Two samples were taken in the dry at each location, one from near the water's edge and the other from the point bar deposits about half the distance to top bank. These samples were sieved separately, and the resulting gradations are plotted on Figures 2 and 3, respectively.

Soils Explorations

13. In 1978 seventeen borings were taken between the mouth and RM 19. Figure 4 presents the locations and soil characteristics. Nine additional borings were taken in 1986. In addition, boring logs are available for the bridge crossings.

Part III: ANALYSIS OF EXISTING CHANNEL

Trends Since 1953

Mosaics from 1953 and 1954

14. Aerial photographs were flown on 6 March 1953 and 16 March 1954 from the mouth to River Mile 3(RM 3). These show a naturally meandering channel undergoing an intensive construction program of channel straightening by cutoffs. In 1953 the channel was about 200 ft wide with a meander wave length of about 2,000 ft between the mouth and RM 1.5. The width decreased, abruptly, and remained about 100 ft for the rest of the mosaic. Backwater from the Mississippi typically extends to RM 2, so this reach of Nonconnah is not considered typical of the project reach. Except for the petroleum facility on McKellar Lake and the large railroad yard near RM 1.5, the land is undeveloped in these photographs.

Mosaic from 1958

15. The earliest aerial mosaic of the entire study reach is from photographs flown in 1958. It shows a rather artificial looking channel up to Winchester Road in that long straight reaches of channel cut through meander traces from recent history. For example, two or three historical meander traces are visible in the tree growth pattern between US Highway 61 Bridges, RM 2.06, and US Highway 51 which crosses at RM 4.46. That meander wave length measures about 1,000 to 1,200 ft. Channel width is on the order of 100 ft. The width estimate is approximate because the scale of the mosaic is 1:24,000. From US Highway 51 to Perkins Road Bridge, RM 11.53, the historical meander wave length is estimated to be 600 to 800 ft. Between Johns Creek, RM 11.94 and Winchester Road, RM 17.34, the meander wave length is estimated to be 400 to 600 ft. With the exception of the airport and a few scattered subdivisions, the area south of Nonconnah Creek is undeveloped within the 3 mile band covered by this mosaic. North of the creek the land use is residential. There are 15 bridge crossings between the mouth and Winchester Road; consequently, the channel alignment was pretty well locked in by 1958.

Mosaic from 1972

16. The next aerial photography is dated 8 August 1972. Interstate Highway 240 (I-240) has been constructed parallel to Nonconnah Creek on the north bank from RM 4 to RM 13. This is the first mosaic after construction of Hickory Hill Road. The land on the south side is urbanized to Hickory Hill Road, RM 14.4. All traces of the historical meander patterns are gone. The overall planform appears relatively stable, but there are reaches which show a tendency to reestablish a meander pattern by building alternate bars in the straight channel alignment. For example, between Airways Blvd., RM 6.79, and Lamar Avenue, RM 8.87, this mosaic shows an alternate bar formation at about 600 to 800 ft spacing which matches the 1958 meander wave length.

Mosaic from 1976

17. This mosaic is from photography dated 22 January 1976. It shows 5 reaches where the Nonconnah Creek channel is being used as a borrow pit. These reaches scale between 500 and 1,000 ft wide, and the total length is 3 miles.

Mosaic from 1986

18. This mosaic is from photography dated 23 January 1986. It shows one new borrow area. Although the stream is highly disturbed by man's activities, the low flow channel shows alternate bar formations which can be used to aid in predicting the meander tendencies in the Corps project channel. In summary, the mosaics indicate a historical channel that was typically meandering, and they show an existing channel which has long straight segments. There are several reaches where the channel is now much wider than the natural channel was. The mosaics also show an alternate bar pattern forming in some existing, straight-channel segments, and those bars have about the same spacing as the historical meander wave length. Once such a trend begins, the bars will usually become larger as time passes, and consequently the flow will be pressed against the opposite bank at each bar. Bank erosion results. The proposed project channel is expected to exhibit a similar behavior, but it will probably progress at the historical rate shown by these mosaics. Therefore, riprap protection will probably be required to maintain the prescribed channel location, but that is expected to be some time in the future. The least expensive method of providing protecting against such bank

erosion is to allow the process to start then riprap a sufficient number of bends on either side to control it.

Reconnaissance of Prototype

Sites Visited

19. A group of hydraulic engineers from Memphis District; Lower Mississippi Valley Division; Office, Chief of Engineers and Waterways Experiment Station visited the following sites along Nonconnah Creek during the afternoon of 10 March 1988:

a. Winchester Road, RM 17.34

b. Mt. Moriah Road, RM 12.59

c. Confluence of Johns Creek, RM 11.50 to 11.94

d. Perkins Road, RM 11.53

e. Getwell Road, RM 10.11

f. Confluence of Ten Mile Creek, RM 9.46

g. US. Highway 51, RM 4.46

h. US Highway 61, RM 2.06 to 2.14

Observations of the Prototype

a. ꞁrom one end to the other there has been so much activity by man that it is difficult to assess the degree of instability of the existing channel. The stream bed profile is degrading as evidenced by gabions, and other types of grade stablization at several bridges. Within the past decade one bridge failure has occurred. Judging by the amount of degradation downstream from the existing grade stablization sites, the degradation trend is still continuing. In general, the banks appear remarkably stable to be so tall and steep.

b. There is evidence that the stream has transported substantial quantities of silt, sand and gravel in the recent past. For example, buried logs were visible 12 to 15 ft below the present top bank. Memphis District

personnel pointed out reaches where local developers had borrowed enough dirt from the Nonconnah Creek channel to fill in the flood plain for an entire shopping mall, and those borrow areas are now being refilled by Nonconnah Creek.

c. A detailed report of bank failure is not possible because the inspection was not continuous over the entire project reach, but rather it included only selected sites. However, there was evidence of bank failure downstream of Getwell Road. Those banks were wet from seepage and the author attributed the cause of failure to geotechnical mechanism.

d. The four locations showing evidence of substantial channel bed degradation were Mt Moriah Bridge, Perkins Road Bridge, the confluence with Johns Creek and the confluence with Ten Mile Creek. These creeks are two of the largest tributaries entering the project reach. Both have drop structures within a few hundred feet of Nonconnah Creek, and the bed has eroded about 8-10 ft downstream from the drop structure.

e. There is evidence of channel aggradation upstream from US 51 Bridge, RM 4.46, and the Illinois Central Gulf Railroad crossing, RM 2.14. The point bar is building and forcing the channel into the bank in the bend approaching these bridges. There is a small gravel mining operation just upstream from the US 51 Bridge.

Bed Profiles

20. Bed profiles from 1967, 1978 and 1987 are shown in Figure 5. Between the 1967 and 1978 surveys the general trend was degradation. Between 1978 and 1987 aggradation is shown between RM 0 and 4, but the overall trend is degradation upstream from there.

Soils Characteristics

21. The soil borings between RM 4.2 and 4.5 show the channel invert to be about the elevation of top of a clay layer, Figure 4. The 3 borings between RM 14 and 17.5 show the existing channel invert from 5 to 8 ft above the top of clay. Those clay layers have substantial thickness. Between RM 4.5 and 14 the present bed is composed of poorly graded sands or gravelly-sands with little or no fines. Samples show silts and clays in the banks above the gravelly-sand layer, but over most of this reach the existing

channel invert has dropped below those silt-clay layers into the gravelly-sands. This shows to be a deep layer. Profiles of D50 sizes of the mid-bar samples and the water's edge samples of bed material are shown in Figure 6.

Analysis of the Streamgage Records

22. The USGS gage is located on the Nonconnah Creek at mile 17.34, the Winchester Bridge. Figure 7 shows a plot of the current USGS rating curve, number 18 dated 1982, plus all measurements since 1980 for water discharges above 1,000 cfs. Also shown on that figure is the rating curve calculated with HEC-2. The calculated values are 3 ft below the USGS curve for a discharge of 8,000 cfs. However, there is reasonable doubt about the validity of the present rating curve. First, for discharges greater than 4,000 cfs the USGS curve is based on 4 measurements made between 1969 and 1980. Secondly, the aerial mosaics show that the reach upstream from Kirby Road was being used as a borrow area in 1976. Finally, the stream bed profiles show up to 3 ft of degradation between Kirby Road and the USGS gage between 1967 and 1986. Consequently, the USGS rating is considered to be less accurate than the calculated water-surface elevations from HEC-2 because the calculations take into account those geometric changes.

Historical Stability of Stage-Discharge Curve

23. There are not a sufficient number of measurements to plot a graph of specific gage height. Therefore, the gage record cannot be used to investigate a trend.

Hydraulic Roughness

24. The most dependable method for determining hydraulic roughness is to reconstitute measured high water profiles from historical floods. However, no field measurements are available on this creek. The second most dependable method is to reconstitute measured gage records, but concerns about them is documented above. Consequently, the recourse is to apply stage-discharge predictors and to use data from other sites via standard, calibrated photographs in Chow's book, Open Channel Hydraulics, to estimate the n value for this project.

Stage-Discharge Predictors

25. The mobile bed portion of the cross section was separated from the bank portion and the bed roughness calculated using mobile bed theory. Brownlie's method predicted an n value range of 0.043 to 0.049 for the movable bed portion of the cross section as the water discharge increased from the 2-year flood peak to the 100-year peak discharge. The White, Paris and Bettis method predicted 0.033. The Limerinos method predicted 0.027. Figure 8 shows the range of channel n values that Memphis District used. Whereas there is not adequate gage data to reconstitute an n value; the bank irregularity, the vegetation in the channel and the Brownlie's method all support the district's choice.

Selection of n Values

26. Manning's n values were seleted after field reconnaissance of the main stem. Flow lines were computed using a Manning's roughness coefficient of 0.040 to 0.060 for existing channels, 0.035 for improved channels, 0.040 for vegetation clearing, and 0.090 to 0.125 for overbank areas. A Manning's n value of 0.035 for improved channels was chosen because of the considerable amount of channel debris and riprap; and the proposed improvement only requires minimal channel alterations in many reaches.

Consequences of Inaccurate n Values

27. In fixed bed hydraulics, a range of n values is typically chosen. The low end of that range provides velocities for riprap design and the high end provides the water-surface profile for flood protection. In movable bed studies there is a strong feed back between sediment transport and hydraulic roughness, and the bed roughness predictor provides that linkage. Manning's n values which do not reflect that linkage either predict too much degradation or too much aggradation.

Hydraulic Parameters

28. Typically velocity, width, depth, slope and meander wave length are expected to be related to a dominant water discharge. The sediment concentration in the flow, sediment particle size in transport, sediment particle size on the stream bed, and cohesive characteristics of the bank material are parameters in those relationships. The 2-year flood peak is

often quoted in the literature as approximating the dominant discharge. Figure 9 shows regime relations for width, depth and slope copied from the report "Hydraulic Design of Stable Flood Control Channels, II - Draft Guidelines for Preliminary Design" which was prepared by Northwest Hydraulic Consultants, LTD, for the Seattle District, US Army Corps of Engineers. Those regime relationships for width, depth and slope are applied to Nonconnah Creek using the 2-year flood peak discharge, 17,692 cfs, at the mouth. This does not imply that Nonconnah Creek is in regime. It does indicate how this channel compares with those known to be in regime.

Calculated Water-Surface Top Width, Existing Conditions

29. Water-surface top widths calculated with HEC-2 are shown on Figure 10. The average width of the existing channel is about 260 ft along the lower 4 miles of channel. This compares favorably with the regime top width from Figure 9 as seen using curve 2:

$$\begin{aligned} W &= 2 * \mathrm{SQRT}(17692) \\ &= 266 \text{ feet.} \end{aligned}$$

Of all reaches this is considered the one most likely to have been formed by alluvial processes because of the lack of recent channel modification by man's activities. The channel cross section has been disturbed upstream from this point, and the calculated top width does not represent a regime value.

Channel Depth

30. The lower portion of Nonconnah Creek is considered by this author to be a very poor location to compare the channel depth to a regime value because the hydraulics-hydrology-sediment transportation characteristics are strongly influenced by the Mississippi River. The best location seems to be one of the upstream borrow areas which is being refilled by Nonconnah Creek.

Energy Slope, Existing Conditions

31. Figure 11 shows a energy slope profile for the 2-year flood as modified by tributary inflows. The energy slope tends to increase slightly in the upstream direction. The mean value between RM 8 and 12 is about 0.0007 ft/ft. Figure 9 shows a regime relationship which depends on the size of bed particles as well as the water discharge. Applying that to Nonconnah Creek between RM 8 and 11, the mean value of D50 is 8 mm and the 2-yr flood

peak is about 9,000 cfs. The resulting regime slope is about 0.00035 ft/ft. Note, this is from the lowest curve on Figure 9 which is for a d50 of 10 mm.

> Since the slope responds more quickly to changes in the inflowing sediment load than does either the channel width or the channel depth, it is the least dependable regime parameter in the opinion of the author. That is, the bed slope depends more strongly on particle size in the stream bed and on concentration of the bed material particle sizes in the inflowing sediment load than does either the regime depth or the regime width. For equilibrium systems in which there is little or no sediment transport, the regime slope is highly significant; but when the stream channel is as far from equilibrium as Nonconnah Creek, one should be interested in but not confined to that slope. On the other hand, the stream bed is degrading along Nonconnah Creek. One would expect that to happen in a channel slope of .0007 if the regime slope were .00035 and the inflowing concentration of sediment was decreasing for the bed-material sizes. Such a decrease seems likely in this creek because of historical mining and extensive urbanization.

Mean Channel Velocity, Existing Conditions

32. The calculated channel velocity for the 2-year peak discharge is shown on Figure 12 for existing conditions. There is no pattern upstream from RM 13, but from RM 13 to the mouth the velocity increases. The values calculated with HEC-2 fluctuate from cross section to cross section, but a regression line through the scatter of points has a value of 3 fps near RM 13 increasing to 6 fps at the mouth.

Inflowing Sediment Load, Existing Conditions

33. No suspended sediment measurements are available, but sands and gravels are the predominant sediment sizes on the bed of the existing channel. Therefore, sediment transport theory was used to calculate the bed material sediment discharge for existing conditions. These calculations require hydraulic parameters plus the gradation of the bed surface. That portion of Nonconnah Creek upstream from Winchester Road, RM 18.10 to RM 20.98, was selected for the transport capacity calculations. The existing conditions geometry and n values formed the geometric model. Four flood discharges were selected, and the starting elevations for the water-surface profiles were taken from the HEC-2 rating curve on Figure 7. TABS-1 was used for the calculations.

Transport Capacity Calculations

34. Since the bed samples from "mid-bar locations" were the most likely to have been deposited during floods, they were used to describe the bed material for sediment transport calculations for the four selected flood discharges. Starting with the 2-year flood peak discharge, a zero sediment inflow was prescribed for the TABS-1 code. The Laursen Transport function as modified by Madden in 1985 was used to calculate the total sand and gravel load moving in the model and the concentration by size class. The average transport capacity was calculated by averaging the 11-cross sectional values for the reach, RM 18.10 to 20.98. Those values were then coded as the inflow to the upstream end of the model and the calculation repeated for that same water discharge. After three iterations, the inflow was in balance with the average transport in that reach as shown by a zero trapping efficiency and negligible bed change at each cross section. That value was selected; the next water discharge was prescribed and the procedure started over. The results of these calculations are the points plotted in Figure 13. The line was extrapolated beyond the calculated points in both directions. Table 2 gives the values, by size class, that were coded into the numerical model.

TABLE 2. The Inflowing Sediment Load by Size Class, Tons/Day(1)

Q(1)	1.000000	8,000.00	16,000.00	100,000.00
VFS	0.002614	100.00	190.00	1,775.00
FS	0.026138	1,000.00	1,900.00	17,750.00
MS	0.018386	703.40	1,336.46	12,485.30
CS	0.0036593	140.00	266.00	2,485.00
VCS	0.0006795	26.00	49.40	461.50
VFG	0.0003136	12.00	22.80	213.00
FG	0.0002404	9.20	17.48	163.30
MG	0.0001934	7.40	14.06	131.35
CG	0.0000522	2.00	3.80	35.50
	---------	--------	--------	---------
SUM	0.0522761	2,000.00	3,800.00	35,500.00

(1) The first value in each column is the water discharge in cfs. The remaining values are the sediment discharges for each size class, listed in column 1, in tons/day.

Sediment Inflow from Tributaries

35. The sediment inflow from tributaries was assumed to be zero. This assumption is not based on data. It was made after a site reconnaissance during which the team looked for a delta at the mouths of the tributaries and it looked for channel bed scour or deposition along the lower end of the tributary. They found no deltas, and there was substantial degradation

downstream from the drop structure on Johns Creek and also on Ten Mile Creek. The drop structures are located a distance of a hundred yards or so upstream from Nonconnah Creek. There was no significant deposits on the concrete lining of those tributaries upstream from the drop structures. These observations supported the assumption that no significant sediment load was coming from the tributaries. Since the 'zero' assumption will lead to more intensive erosion on the main stem, it is justified at this point in the project design.

Assessment of Existing Channel

36. Based on the aerial mosaics, the reconnaissance trip, historical bed profiles, samples of the bed material, soils boring logs, and the historical mining of channel sediments, the existing channel is judged to be unstable. That means that bed degradation is expected to continue at the historical rate or perhaps greater unless some corrective action is taken to stablize the channel invert. If no corrective action is taken, the bank heights are expected to exceed the strength of the bank sediments, and bank failures will increase in size and number within the 50-year life of the proposed project. Moreover, bank erosion is expected to increase as the alternate bar formations continue to develop and grow in height.

Part IV: BASE TEST

Description of the Numerical Model

Model Geometry and the Bed Sediment Reservoir

37. The TABS-1 model geometry has 143 computation cross sections formed from the HEC-2 model for this project. It was developed by replacing the 4 cross sections HEC-2 uses to describe hydraulic losses at a bridge with one through the throat of the bridge. Figure 14 shows 43 cross section locations. These are the Corps of Engineers sections. Of the 71 cross sections surveyed by the Corps of Engineers in 1986, Twenty eight are bridges sites and two sections were surveyed at each site. Of the 72 cross sections remaining, 69 were surveyed by others, and three are repeated sections. The bed sediment reservoir extends the width of the channel and the depth was arbitrarily limited to 10 ft except at bridges where gabions, or some other scour resistant material, has been placed. A zero depth of bed was assigned at those hard points.

Boundary Conditions

38. The water discharge hydrograph was developed for River Mile 0.0 on the main stem by calculating runoff using historical rainfall records. This step was necessary so the relative impact, on the runoff hydrograph, of future changes in land use could be estimated. Years 1964-1987 from the rainfall station at Memphis International Airport were used in this calculation. The rainfall during that historical period is assumed to be representative of future rainfall events. That is a common assumption.

39. Composite unit hydrographs were developed from the 10-year and the 100-year flood hydrographs that were produced by HEC-1 for the Phase II General Design Memorandum. The composite unit hydrographs were then adjusted to a 24-hour duration and a 6-hour duration, respectively, using the S-curve method. Memphis District used its computer program to combine the historical rainfall data file with that adjusted unit hydrograph using the antecedent precipitation index method to compute losses. The resulting annual peak discharges are plotted on Figures 15 and 16 to show they are consistent with the historical record. The discharge statistics were estimated by converting the historical stage record at the Winchester gage into one discharge record

for existing conditions and another for project conditions using the rating curves calculated with HEC-2.

40. These results were converted into two histograms, one for historical and the other for future conditions, using the Sediment Weighted Histogram Generator (SWHG) developed by the Hydrologic Engineering Center, Davis, California. As an example, Figure 17 shows the histogram for historical flows on the main-stem at RM 0. This utility program processes mean daily flows and develops a histogram that aggregates the daily events into representative discharges spanning longer time periods. The histograms were then transposed to three locations within the study area using total water volume as the basis for proportioning tributary inflows.

41. Tributaries were grouped at the points shown in Table 3, and the inflowing water discharge histographs were calculated from the main stem histographs. Table 3 also shows the inflow distribution coefficients used to calculate the tributary inflows. The 2-year flood peak is used for the example, but the same coefficients were used for all flows.

TABLE 3. Distribution of Runoff by Tributary, 2-Year Flood Peak

River Mile	Qmain cfs	Qtrib cfs	f-main	f-trib
0 - 7.903	17,692			
		4,044	.7714	.2286
7.903 - 11.942	13,648			
		4,777	.6500	.2699
11.942 - 17.346	8,871			
		1,696	.8088	.0958696
17.346 - 21.005	7,175			

The resulting water discharge profile is shown on Figure 18. The coefficient f-main was determined from the HEC-1 analysis of the watershed. It is the ratio of the main stem flow upstream from the tributary to that downstream from it. The coefficient f-trib was calculated from f-main and is the ratio of the tributary discharge to Qmain at RM 0. Memphis District personnel developed these coefficients and used them in distributing the histogram flows by tributary. Tributary inflows are calculated with the coefficients shown in Table 3.

42. The third boundary condition is the inflowing sediment concentration by particle size fraction, shown in Table 2. These values assume the current sediment concentrations and particle sizes will not change in the future, an assumption which seems reasonable in this case.

43. The final boundary condition is the stage-discharge rating curve at RM 0, the outflow Boundary. That is a normal depth rating furnished by Memphis District, Figure 19. This does not capture the backwater condition when the Mississippi River is high; therefore, the model behavior downstream from approximately RM 2 is qualitative and describes a necessary but not sufficient number of conditions to test the design. Another condition would be described by a stage hydrograph of the Mississippi River for the same period as the rainfall records, but those data were not available for this study.

Results

Predicted Bed Surface Profiles

44. One measure of stream bed stability is the calculated aggradation and degradation of the stream bed profile. Figure 20 shows the existing bed surface profile and the one calculated for the end of 24 years. Because the scale makes this figure difficult to interpret, the calculated changes in bed elevation are shown in Figure 21. Positive values mean deposition (aggradation), and negative mean erosion (degradation). Because of the influence of Mississippi River backwater, the lower 2 miles are not representative of the long term trend on Nonconnah Creek. The predicted erosion ranges up to 9 ft downstream from Mt Moriah Bridge with deposition zones up to 4 ft in the reach between Kirby Road and Winchester Bridge. However, most of the 20 mile reach shows a degradation trend.

Predicted Water-Surface Profiles

45. Figure 22 shows the calculated water-surface profile for the existing channel and for the end of the 24 year period for a relatively low flow of 1,000 cfs. The low flow profile was chosen over the design flood profile because the TABS-1 computer program does not calculate head-loss at bridges like HEC-2 does, and the results could have been misleading. The difference between the existing water-surface and that at year 24 is the amount the main stem water-surface profile will be lowered because of the degrading streambed if no project is constructed. This is referred to as "base level lowering."

The water-surface elevation on Nonconnah Creek controls all tributary water-surfaces. The term "base level lowering" means that which controls the basic rate of energy dissipation in the stream, i.e. the base level, will become lowered in elevation as time passes. That is very significant because as the base level becomes lower tributary energy gradients will increase possibly causing severe erosion and head cuts to migrate up the tributary. Each tributary entry point has to be designed to prevent such from occurring.

Bank Stability

46. The computer calculations do not address bank stability. However, several reaches show more than 6 ft of bed degradation. This does not mean degradation will stop after 24 years, the degradation trend is continuing. Consequently, banks are expected to become too high for the material to remain stable at the steep bank angles common to this process, and geotechnical failures will likely occur. Stability needs to be evaluated, but that is beyond the scope of this study.

Part V: PROJECT CONDITIONS

Description of the Numerical Model

Model Geometry and the Bed Sediment Reservoir

47. The existing cross sections were modified to include the design channel cross section. Hard points were inserted at bridge and pipeline crossings. Hydraulic roughness was changed to reflect project conditions.

Boundary Conditions

48. All boundary conditions are the same as described for the Base Test.

Results

Predicted Bed Surface Changes

49. The calculated bed surface profile at the end of the 24-year forecast period is shown on Figure 20 along with the existing channel and the Base Test profiles. Existing hard points are shown as circles on the bed profile. These are local protection at bridge crossings. Note that between RM 3 and RM 12 the future bed profile is substantially lower with the project than without. That reflects the design of the project channel and not degradation due to stream bed erosion.

50. Figure 21 shows the calculated bed aggradation/degradation plotted as bed change versus river mile. The Base Test is the prediction of future stream bed aggradation and degradation if the Corps of Engineers project is not constructed. The Plan Test is an identical forecast assuming the project is in operation. Bed change values are generally closer to 'zero' with the project than without it. That demonstrates the channel bed will be more stable with the project than without it.

51. To illustrate the point, Table 4 shows TABS-1 results at each cross section. The first column is the cross section location by river mile. The second column is the calculated bed change for the Base Test and the third column is the calculated bed change for the Plan Test. These columns repeat across the page. Table 5 is a summary of those values by project reach. River mile 0 to 18.2 is the project reach; however, because this study did not

TABLE 4. Calculated Change in Bed Elevation, 24-Year Forecast

Section Id No	Base Test Ft	Plan Test Ft	Section Id No	Base Test Ft	Plan Test Ft	Section Id No	Base Test Ft	Plan Test Ft
21.005	1.1	1.1	12.210	1.1	1.3	6.560	0.4	-1.6
20.980	0.0	0.9	12.144	-0.4	-0.8	6.490	0.0	0.0
20.791	-1.0	-0.5	12.018	-1.0	-1.1	6.301	-1.6	-0.1
19.870	0.7	1.2	11.942	0.0	-0.1	5.920	1.1	0.4
19.690	1.2	1.9	11.896	-1.9	-2.8	5.621	0.8	0.6
19.520	1.3	1.6	11.840	-3.1	-4.6	5.540	0.1	0.0
19.066	0.3	0.4	11.783	-4.0	-2.9	5.250	-4.5	-2.6
18.971	0.0	0.0	11.726	0.4	-1.7	4.960	-4.5	-1.3
18.895	-0.4	-0.1	11.669	-0.9	-1.4	4.491	-4.4	-4.9
18.850	-1.2	0.0	11.612	-2.5	-2.8	4.472	-4.2	0.0
18.600	-1.9	-2.5	11.555	-6.3	-7.8	4.460	-5.8	0.0
18.100	-0.2	-1.9	11.530	0.0	0.0	4.349	-3.8	-2.3
17.365	0.0	-0.6	11.498	0.2	-4.2	4.320	-4.3	0.0
17.346	-1.5	0.0	11.435	2.2	1.1	4.311	0.1	0.4
17.340	-2.7	0.0	11.230	1.7	0.8	4.170	-0.2	-0.8
17.246	-2.1	-1.3	11.006	-0.8	-2.8	4.161	0.6	0.0
17.113	-3.3	-3.7	10.869	1.4	0.2	4.140	1.1	0.0
16.670	-2.8	-2.7	10.727	1.1	-0.1	4.131	1.1	-0.6
16.400	-3.4	-3.8	10.695	1.1	1.0	3.639	-3.1	0.0
16.347	4.4	4.7	10.637	1.0	0.8	3.450	-3.6	0.3
16.252	0.2	0.2	10.527	0.5	-0.4	3.380	-4.1	0.5
16.195	-2.6	-1.6	10.461	-1.1	-0.8	3.225	-1.8	0.3
15.840	1.1	1.0	10.351	-2.9	-0.7	3.135	-4.7	0.3
15.624	-1.3	-0.6	10.256	-3.1	-1.5	3.048	-4.1	0.4
15.605	-1.5	0.0	10.167	-4.2	-4.8	2.650	0.1	0.4
15.600	-4.6	0.0	10.129	0.0	0.0	2.590	-2.6	0.0
15.525	-3.9	-3.8	10.110	0.0	0.0	2.350	-4.5	0.0
15.431	-3.6	-6.7	9.880	-2.1	-4.7	2.184	-6.0	-4.2
15.336	-0.7	-4.3	9.830	-3.0	-3.1	2.140	-7.8	0.0
15.260	-3.1	-3.8	9.450	0.0	-2.9	2.088	-3.4	0.0
15.190	-1.9	0.3	9.100	-1.1	-0.7	2.079	-9.3	0.0
15.095	-1.2	-0.3	8.926	-1.5	-1.7	2.070	-9.4	0.0
15.000	0.0	-0.3	8.888	-1.0	0.1	2.060	-6.1	0.0
14.860	-2.0	-1.9	8.870	-1.6	0.1	2.051	-3.1	-7.4
14.457	-2.0	-2.8	8.610	-0.9	0.5	1.833	-1.7	-4.7
14.419	0.0	0.0	8.497	-0.6	0.0	1.786	-3.2	-4.4
14.400	0.0	0.0	8.490	-0.9	0.0	1.760	-3.9	-5.2
14.370	-1.4	-2.8	8.180	1.8	0.9	1.751	-4.0	-5.3
14.211	-1.7	-2.1	8.092	0.8	0.0	1.690	-4.0	-5.5
14.040	0.0	-1.2	7.960	0.8	-0.7	1.661	-4.1	-5.2
13.600	-3.5	-3.0	7.903	0.5	-0.9	1.481	-4.0	-4.2
13.380	-2.8	-2.1	7.780	-1.3	-2.3	1.230	-4.2	-5.0
13.010	0.8	1.0	7.625	-4.6	-2.6	0.797	-1.4	-2.9
12.632	-1.6	-0.6	7.220	-4.9	-3.0	0.760	0.2	-1.5
12.613	0.0	0.0	6.899	-2.3	-1.7	0.750	-3.3	-3.8
12.590	0.0	0.0	6.862	-1.4	-1.1	0.617	-4.5	-5.3
12.461	-9.8	-9.8	6.850	-2.0	0.0	0.290	-2.9	-3.1
12.308	2.6	2.5	6.740	-1.7	0.0			

TABLE 5. Summary of erosion and deposition by reach

River Mile	Base Test erosion ft	Base Test deposition ft	Plan Test erosion ft	Plan Test deposition ft
2.180 - 7.780	-3.4	0.5	-1.2	0.3
7.903 - 8.497	-1.8	0.9	-2.0	0.2
8.610 - 10.351	-1.8	0	-2.0	0.2
10.461 - 18.100	-2.0	1.2	-2.0	1.2
18.600 - 21.005	-0.9	0.8	-0.8	1.0

include the Mississippi River backwater, the calculations are not reliable downstream from RM 2. River Mile 18.2 to 21 is the approach channel. In the Base Test the largest depth of degradation, 3.4 ft, occurs between RM 2.184 and 7.780. That is reduced to 1.2 ft in the Plan Test because numerous hard points were added in this reach.

> (Note: Calculated results are shown to the tenth of a foot to aid in relating conclusions back to the calculations and does not imply the accuracy of results.)

Between RM 7.780 and 8.497 the channel cross section will not be modified. However, Manning's n value is reduced by removing vegetation which, coupled with the lower water surfaces downstream from RM 7.780, will increase bed degradation in this reach from 1.8 ft in the Base Test to 2.0 ft in the Plan Test. That is an increase of 10 percent and is considered tolerable. That trend persists to RM 10.351, but Base and Plan tests gave the same results upstream from RM 10.351.

52. Of particular interest is the annual rate of aggradation and degradation of the bed elevation. Figure 21 shows a great deal of variation from river mile to river mile at the end of 24 years, and no cross section seems to be "representative" of the entire model. Therefore, searching for the more dramatic cases led to the selection of the two cross sections having the largest amount of degradation. Figure 23 shows a bed change versus time plot for RM 12.461. This is just downstream from Mt Moriah Road.

> Note: The total time in the 24-year flood hydrograph is 2,500 days because the stream discharge hovers around zero for extended periods between the flood events. These were omitted from the simulation because no significant degradation changes occur at the extremely low flows.

By the end of year 5, which is day 497.86 in Figure 23, the bed has degraded 6 ft. There is a dramatic reduction in the rate of degradation over the next 19 years. That reduction is not due to hydrology, shown by the hydrograph in Figure 18. It could be the decrease in energy slope or it could be an increase in resistant forces because the percent of coarse particles increases as the fines are transported away. Dividing the total degradation by 24 years gives a rate of about 0.5 ft/year.

53. The calculated bed elevation change versus time is also shown for RM 4.32 in Figure 24. This location, between I-55 and US Highway 51, showed the second largest amount of degradation.

Predicted Water-Surface Profiles

54. The calculated water-surface profile at the end of the 24-year forecast period is shown on Figure 22 for a low flow water discharge of 1,000 cfs. Figure 25 shows the change in water-surface elevation versus time at RM 12.461 during the 24-year forecast period. Plotted on that same figure is the change in the 100-yr flood elevation. That is the TABS-1 estimate, and not the HEC-2 calculation. It shows changes due to aggradation and degradation of the channel bed. Figure 26 is a similar plot for RM 4.32.

Base Level Lowering

55. As the bed degrades, the water-surface in Nonconnah Creek will drop causing steeper gradients on tributaries. Between RM 3 and 12, the reach where channel modification is proposed, the project will lower the base-level up to 6 ft more than is predicted for the no project condition. Table 6 shows the location by River Mile in column 1. The water-surface profile for initial conditions (T0) is shown in column 2; the predicted water-surface profile at 24-years in the future for the Base Test in column 3 (T24); and the predicted water-surface profile at 24-years in the future for the Plan Test is in column 4 (P24). The water discharge for these results is 1,000 cfs at the mouth decreasing to 370 cfs upstream from RM 17.340. Therefore, not only must bridges and pipe line crossings be protected on the main stem, but also the tributary streams and local runoff structures must be protected against the increased erosive forces due to base level lowering.

Bank Stability

56. The project reach upstream from the modified channel portion should be evaluated for bank stability in view of the layering of materials in the bank and the additional bank height that is expected as the result of channel bed lowering. That condition is expected to occur without the project, but it should be evaluated to ascertain that the project does not make it worse.

The Approach Channel

57. Upstream from the project reach, the calculations show negligible base level lowering. Consequently, the approach channel has been adequately protected against future degradation from the project.

TABLE 6. Base Level Lowering With and Without the Project

Section Id No	Initial WS Elev (T0)	Predicted WS Elev (T24)	Predicted WS Elev (P24)	Base Level Change T24-T0	Base Level Change P24-T0
21.005	289.18	289.54	289.51	0.36	0.33
20.980	289.05	289.11	289.06	0.06	0.01
20.791	287.60	287.66	287.42	0.06	-0.18
19.870	282.20	284.12	283.90	1.92	1.70
19.690	281.44	283.42	283.24	1.98	1.80
19.520	280.75	282.42	282.25	1.67	1.50
19.066	279.04	279.11	278.84	0.07	-0.20
18.971	278.75	278.51	278.34	-0.24	-0.41
18.895	278.35	277.73	277.66	-0.62	-0.69
18.850	278.11	277.10	276.81	-1.01	-1.30
18.600	276.87	275.39	274.65	-1.48	-2.22
18.100	274.45	273.42	272.35	-1.03	-2.10
17.365	270.95	269.68	270.72	-1.27	-0.23
17.346	270.82	269.47	270.42	-1.35	-0.40
17.340	270.69	269.44	270.31	-1.25	-0.38
17.246	270.13	268.64	268.18	-1.49	-1.95
17.113	269.36	268.03	267.81	-1.33	-1.55
16.670	266.79	267.48	267.59	0.69	0.80
16.400	264.83	267.35	267.53	2.52	2.70
16.347	264.84	267.15	267.33	2.31	2.49
16.252	264.80	264.68	264.01	-0.12	-0.79
16.195	264.62	264.42	263.85	-0.20	-0.77
15.840	263.84	262.84	262.41	-1.00	-1.43
15.624	263.63	261.31	261.84	-2.32	-1.79
15.605	263.50	261.21	260.73	-2.29	-2.77
15.600	263.50	261.22	259.88	-2.28	-3.62
15.525	263.33	261.16	259.77	-2.17	-3.56
15.431	262.50	260.93	259.70	-1.57	-2.80
15.336	261.40	260.26	259.63	-1.14	-1.77
15.260	260.39	259.23	259.37	-1.16	-1.02
15.190	259.76	259.01	258.96	-0.75	-0.80
15.095	259.36	258.82	258.49	-0.54	-0.87
15.000	258.99	258.44	258.08	-0.55	-0.91
14.860	258.69	258.23	257.96	-0.46	-0.73
14.457	258.03	258.01	257.84	-0.02	-0.19
14.419	257.83	257.87	257.71	0.04	-0.12
14.400	256.80	256.82	256.83	0.02	0.03
14.370	254.47	253.76	252.32	-0.71	-2.15
14.211	254.10	253.52	252.14	-0.58	-1.96
14.040	253.56	252.75	251.53	-0.81	-2.03
13.600	250.74	248.28	248.27	-2.46	-2.47
13.380	248.53	247.69	247.81	-0.84	-0.72
13.010	247.11	246.50	246.88	-0.61	-0.23
12.632	245.97	245.41	245.61	-0.56	-0.36
12.613	245.85	245.20	245.39	-0.65	-0.46
12.590	245.78	243.96	243.89	-1.82	-1.89
12.461	244.35	242.73	242.97	-1.62	-1.38
12.308	241.11	242.07	242.35	0.96	1.24
12.210	240.99	241.14	241.08	-3.85	0.09
12.144	240.90	240.42	240.26	-0.48	-0.64
12.018	240.56	239.42	239.23	-1.14	-1.33
11.942	240.41	238.96	238.74	-1.45	-1.67
11.896	240.12	238.89	238.69	-1.23	-1.43
11.840	239.55	238.84	238.66	-0.71	-0.89
11.783	239.22	238.82	238.63	-0.40	-0.59
11.726	239.17	238.77	238.61	-0.40	-0.56
11.669	239.08	238.72	238.57	-0.36	-0.51
11.612	238.95	238.69	238.55	-0.26	-0.40
11.555	238.75	238.68	238.55	-0.07	-0.20
11.530	237.75	238.30	237.77	0.55	0.02
11.498	236.99	238.24	237.19	1.25	0.20
11.435	236.97	238.07	236.99	1.10	0.02
11.230	236.85	237.18	235.92	0.33	-0.93
11.006	236.42	236.59	235.34	0.17	-1.08
10.869	236.24	236.29	235.04	0.05	-1.20
10.727	236.13	235.96	234.51	-0.17	-1.62

Section Id No	Initial WS Elev (T0)	Predictd WS Elev (T24)	Predictd WS Elev (P24)	Base Level Change T24-T0	Base Level Change P24-T0
10.695	236.12	235.92	234.28	-0.20	-1.84
10.637	236.10	235.88	233.95	-0.22	-2.15
10.527	236.05	235.78	233.32	-0.27	-2.73
10.461	236.00	235.73	233.02	-0.27	-2.98
10.351	235.88	235.68	232.73	-0.20	-3.15
10.256	235.80	235.66	232.67	-0.14	-3.13
10.167	235.70	235.65	232.66	-0.05	-3.04
10.129	235.35	235.38	232.01	0.03	-3.34
10.110	234.97	234.19	230.34	-0.78	-4.63
9.880	231.91	230.43	226.76	-1.48	-5.15
9.830	231.56	230.33	226.68	-1.23	-4.88
9.450	230.21	229.42	226.20	-0.79	-4.01
9.100	229.03	227.55	225.65	-1.48	-3.38
8.926	228.27	226.76	225.44	-1.51	-2.83
8.888	228.02	226.33	225.24	-1.69	-2.78
8.870	227.84	226.07	225.12	-1.77	-2.72
8.610	225.14	224.69	224.05	-0.45	-1.09
8.497	223.27	224.16	223.44	0.89	0.17
8.490	223.02	224.09	223.30	1.07	0.28
8.180	221.74	221.05	219.72	-0.69	-2.02
8.092	221.66	220.39	218.75	-1.27	-2.91
7.960	221.58	219.60	217.30	-1.98	-4.28
7.903	221.56	219.33	216.65	-2.23	-4.91
7.780	221.35	219.08	216.32	-2.27	-5.03
7.625	220.79	218.89	216.07	-1.90	-4.72
7.220	219.73	218.69	215.82	-1.04	-3.91
6.899	218.90	218.50	215.67	-0.40	-3.23
6.862	218.78	218.44	215.64	-0.34	-3.14
6.850	218.74	218.44	215.49	-0.30	-3.25
6.740	218.39	218.33	212.64	-0.06	-5.75
6.560	217.60	217.88	211.89	0.28	-5.71
6.490	216.88	216.24	211.38	-0.64	-5.50
6.301	214.41	214.09	209.59	-0.32	-4.82
5.920	213.32	213.04	207.70	-0.28	-5.62
5.621	212.93	211.80	205.96	-1.13	-6.97
5.540	212.79	210.76	204.71	-2.03	-8.08
5.250	211.98	210.03	203.91	-1.95	-8.07
4.960	210.91	209.83	203.55	-1.08	-7.36
4.491	209.61	209.65	203.43	0.04	-6.18
4.472	209.60	209.65	203.22	0.05	-6.38
4.460	209.59	209.65	203.12	0.06	-6.47
4.349	209.44	209.62	203.02	0.18	-6.42
4.320	209.40	209.59	202.84	0.19	-6.56
4.311	209.36	209.56	202.80	0.20	-6.56
4.170	208.94	209.23	202.36	0.29	-6.58
4.161	208.94	209.14	202.10	0.20	-6.84
4.140	208.91	208.74	201.66	-0.17	-7.25
4.131	208.89	208.61	201.67	-0.28	-7.22
3.639	207.65	204.26	199.28	-3.39	-8.37
3.450	205.90	202.19	198.46	-3.71	-7.44
3.380	204.34	201.48	198.13	-2.86	-6.21
3.225	203.88	199.85	197.60	-4.03	-6.28
3.135	203.26	199.25	197.42	-4.01	-5.84
3.048	200.31	198.91	197.29	-1.40	-3.02
2.650	198.45	195.22	197.06	-3.23	-1.39
2.590	198.33	194.79	196.92	-3.54	-1.41
2.350	197.94	194.42	196.80	-3.52	-1.14
2.184	197.53	194.23	196.77	-3.30	-0.76
2.140	197.27	194.20	196.49	-3.07	-0.78
2.088	197.21	194.16	196.37	-3.05	-0.84
2.079	197.20	194.15	196.35	-3.05	-0.85
2.070	197.18	194.13	196.33	-3.05	-0.85
2.060	196.94	194.10	195.02	-2.84	-1.92
2.051	196.92	194.07	191.91	-2.85	-5.01
1.833	196.37	192.83	191.59	-3.54	-4.78
1.786	196.25	192.42	191.42	-3.83	-4.83

Section Id No	Initial WS Elev (T0)	Predicted WS Elev (T24)	Predicted WS Elev (P24)	Base Level Change T24-T0	Base Level Change P24-T0	Section Id No	Initial WS Elev (T0)	Predictd WS Elev (T24)	Predictd WS Elev (P24)	Base Level Change T24-T0	Base Level Change P24-T0
1.760	196.13	192.26	191.30	-3.87	-4.83	0.797	189.73	186.20	185.35	-3.53	-4.38
1.751	196.08	192.22	191.28	-3.86	-4.80	0.760	189.68	185.60	184.92	-4.08	-4.76
1.690	195.82	191.97	191.14	-3.85	-4.68	0.750	189.59	185.36	184.70	-4.23	-4.89
1.661	195.69	191.84	191.07	-3.85	-4.62	0.617	189.06	184.22	183.76	-4.84	-5.30
1.481	194.83	190.75	190.10	-4.08	-4.73	0.290	182.93	182.06	182.06	-0.87	-0.87
1.230	191.80	189.51	188.43	-2.29	-3.37						

The Exit Channel

57. This model is not accurate in the exit channel because the actual Mississippi River stages were not utilized for the downstream boundary. It would require silt and clay inflows plus an extension of the model limits in the downstream direction. This study was designed to evaluate, primarily, degradation in the project reach.

Part VI: SINGLE EVENT ANALYSIS

The Model

59. Both the Base Test and Plan 1 models were run with the 100-year hydrograph boundary conditions. The inflowing sediment load and the tailwater rating curve developed for the 24-year forecast were used here, also. The 100-year flood hydrograph was converted into the histograph, shown in Figure 27.

Results

Predicted Bed Elevation Changes, 100 Year Flood

60. The calculated changes in bed surface elevation are shown in Figure 28 for both the Base Test and the Plan Test. The maximum bed change for the Base Test coincided with the flood peak, 42,711 cfs. The calculated amount was about 4 ft and occurred near the I-55 bridges. However with the proposed project, the calculated maximum erosion was less than a foot in that vicinity. The maximum erosion was 3 ft and occurred between Perkins Road Bridge and Johns Creek, RM 11.53 and 11.94, 12 hours after the peak of the hydrograph. Figure 28 shows three locations where the predicted bed erosion or deposition with the proposed project is larger than that predicted for the Base Test. These are RM 6 to 8; 11 to 13 and 14 to 16. Except for the Perkins Road to Johns Creek reach, these changes are all less than a foot. Figure 29 is a graph of the calculated bed elevation changes at the end of the flood. Notice the deviations display the same pattern as Figure 28, but the maximums are less which indicates channel filling, and reworking of bed deposits, on the recession side of the hydrograph.

Calculated Change in Water-Surface Profiles, 100 Year Flood

61. Because the TABS-1 computer program does not calculate special bridge losses, water-surface profiles for the Base Test and Plan Test are not displayed. However, Figure 30 shows the difference between Base and Plan. Negative values show where the water-surface is lower with the proposed project than it would be without the project. Positive values show where the water-surface elevation with the project will be higher than would have occurred without the project.

PART VII: CONCLUSIONS AND RECOMMENDATIONS

Assessment of the Proposed Channel

62. The proposed channel design consists of various lengths of vegetative clearing and/or trapezoidal channel with a 100 ft bottom width and depth. The top width of the modified channel approximates that of the existing channel sufficiently close to be a satisfactory design. The depth of the existing channel is deeper than regime depth. When that happens the channel portion of the cross section will convey more of the total flood discharge than normal in a regime channel and that allows flood discharges more potential to erode the bed. The design channel does not decrease that potential; neither does it make it worse. The design does incorporate hard points to increase resistance to erosion. The design channel passed the design flood without excessive erosion. That indicates it would also pass the more frequent floods satisfactorily. The design slope is very close to the historical value. There are a few locations where the alignment of the design channel shows excavation of both banks. Strong consideration should be given to realignment so only one bank requires excavation. A flat bottom trapezoid is not a good design from the standpoint of flow pattern and fish productivity. Fluctuations in water depth such as provided by pools and crossings is needed.

63. Although the project channel shows a degradation trend, this study supports the district's position that the project will make a more stable Nonconnah Creek than will exist without the project. That is, the calculated maximum amount of degradation is about the same with the project as without it. However, the average amount of degradation over the 18.2 mile project length is 1.7 ft with the project and 2.4 ft without it for the 24-year period of this analysis.

(Note: Calculated results are shown to the tenth of a foot to aid in relating conclusions back to the calculations and does not imply the accuracy of results.)

This is a 30 percent reduction. The average amount of aggradation is 0.9 ft without the project and 0.7 ft with it for a reduction of 32 percent. On the other hand calculations show the degradation and aggradation trends will continue beyond the 24-year forecast period at no decrease in rate. That is, in 50 years the average depth of degradation is expected to be 3.4 ft with the

project and 4.8 ft without it. The sponsor should be made aware that such continued downcutting of the stream bed, either with or without the project, will eventually increase bank heights to the point of failure. If that should be allowed to happen, Nonconnah Creek would begin a widening process that would be very expensive to control. The project should be monitored at established sediment ranges to detect such a condition and plans implemented to prevent it from occurring.

64. Whereas the amounts of general aggradation and degradation seem manageable, local erosion, illustrated in Figure 23, can be both rapid and extreme. Typical zones are at and downstream from contractions. These should be monitored.

Assessment of Conditions in the Approach Channel

65. The calculated degradation in the approach channel is reduced about 10% with the proposed project. This is attributed to the hard points proposed at the bridge crossings for Winchester Road, Hacks Cross Road and Forest Hill Road. The calculated water-surface profiles show no appreciable base level lowering in the 3 mile long approach channel to the project.

Assessment of Conditions in the Exit Channel

66. Flows from Nonconnah Creek exit through McKellar Lake to the Mississippi River. No calculations were made in that reach, but the district has dredged there as well as in the lower end of Nonconnah Creek. Maintenance records indicate the dredged volume has been perhaps 2 to 3 times the calculated sand and gravel discharge from Nonconnah Creek in this study. Conditions can be inferred from the calculated Plan Test that the proposed project would decrease sand and gravel outflow by 28 percent. This would be a reduction in a major sediment source to the lake, and that is expected to reduce maintenance dredging quantities for that portion of the navigation project. All concerned should be made aware that the project is not causing such a condition, however the proposed design does not stabilize the creek to the point of preventing such a condition from occurring.

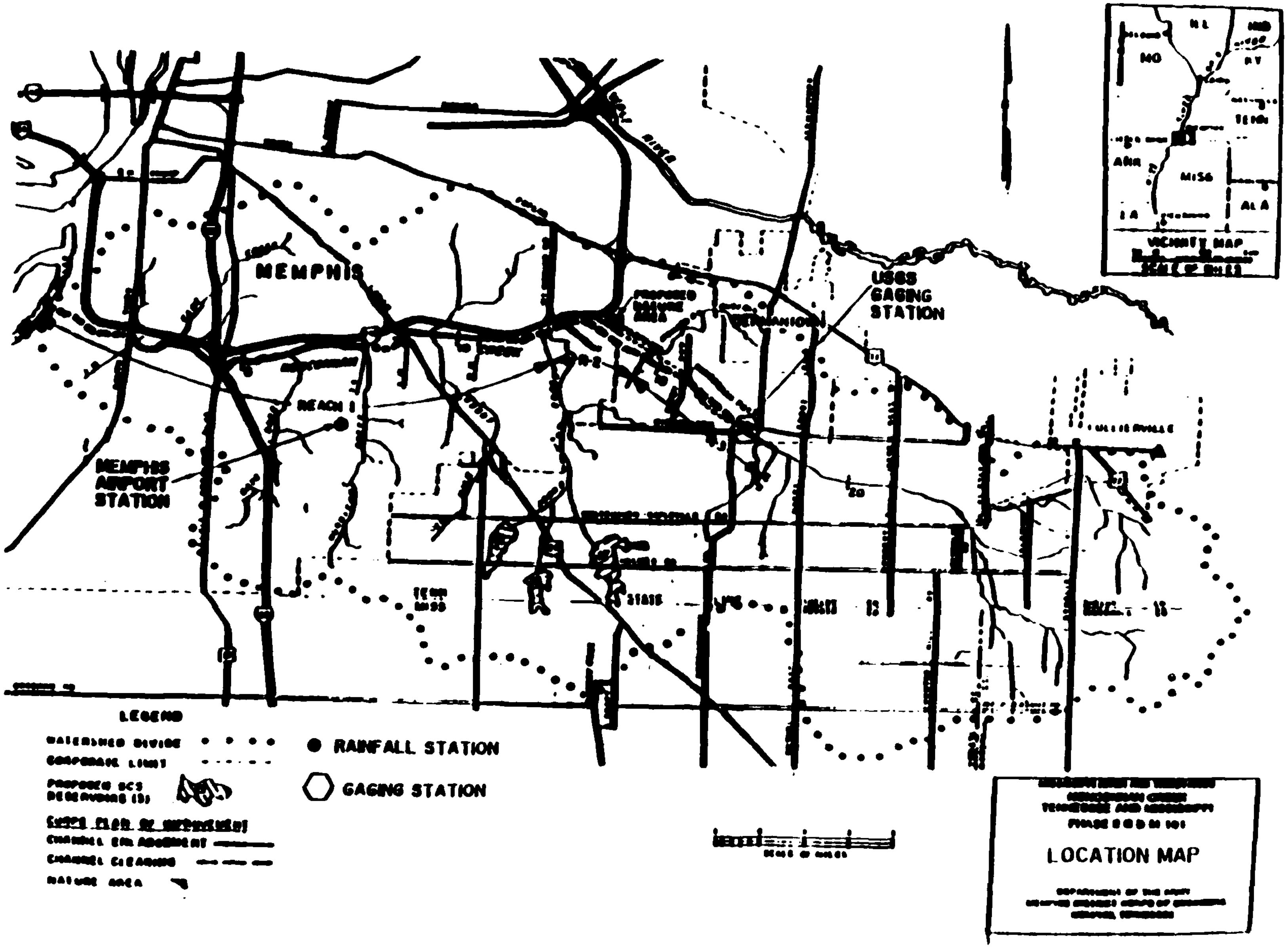


Figure 1. Location map

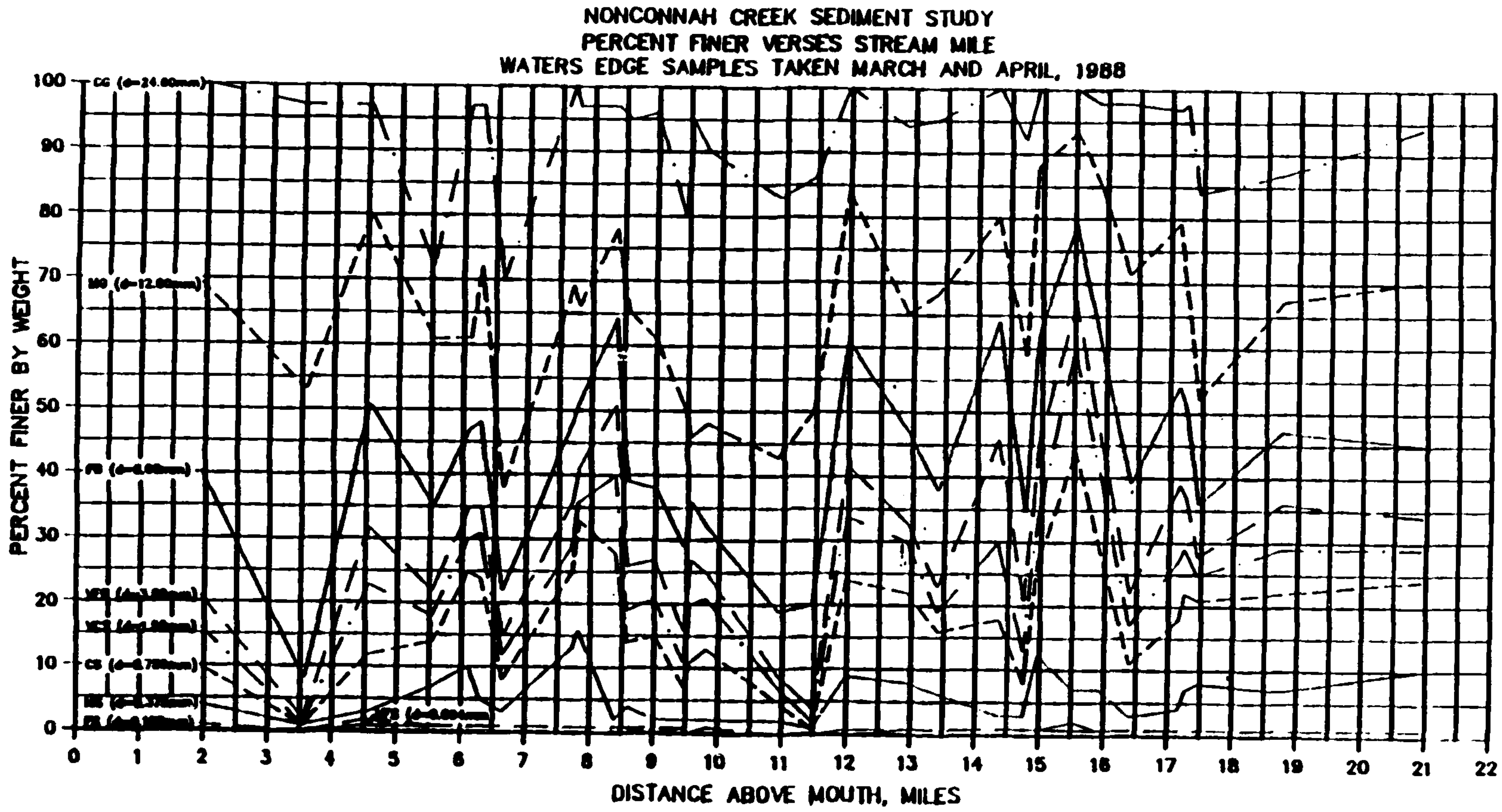


Figure 2. Gradation of bed based on water edge samples

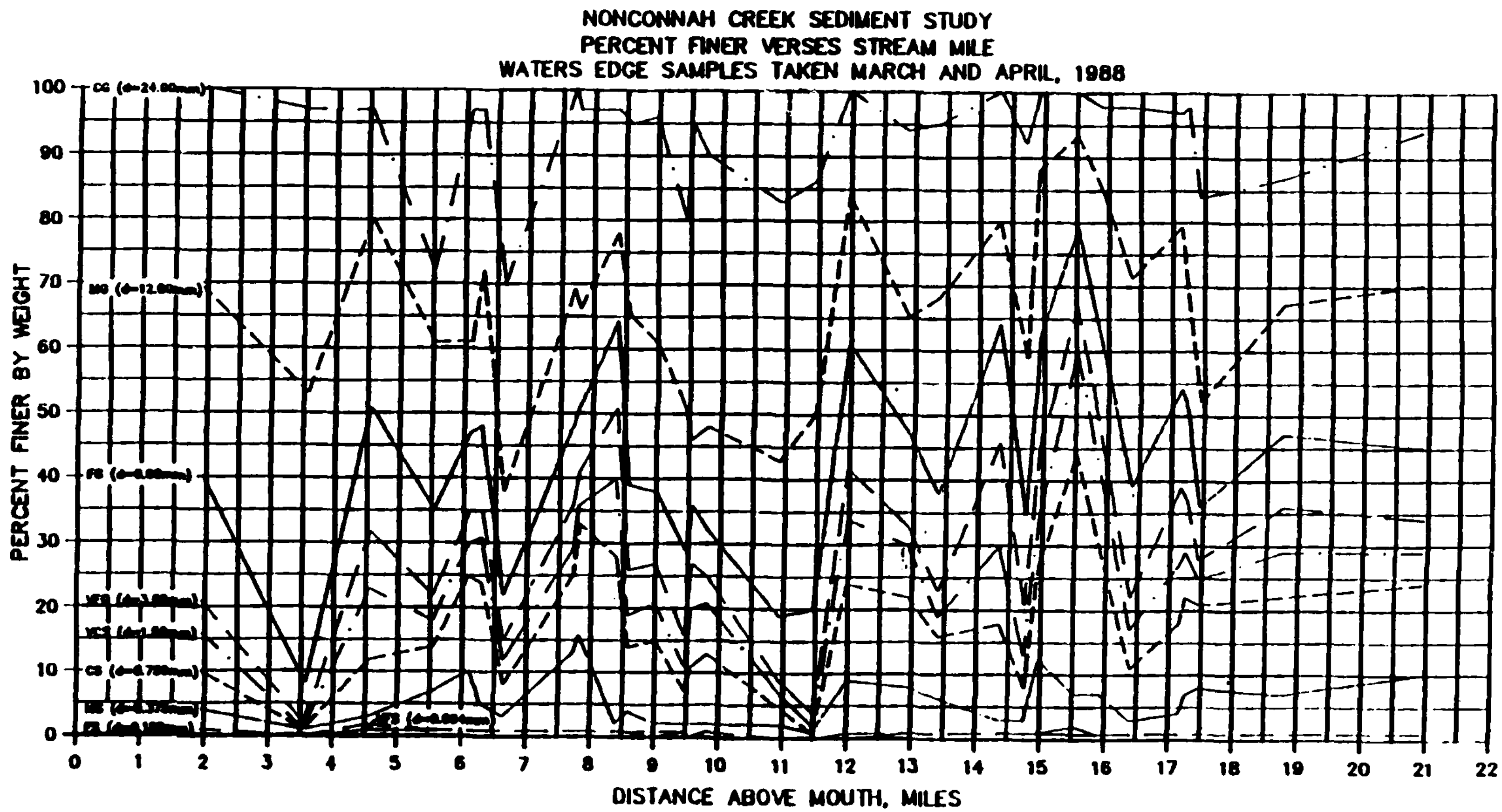


Figure 3. Gradation of bed based on mid-bar samples

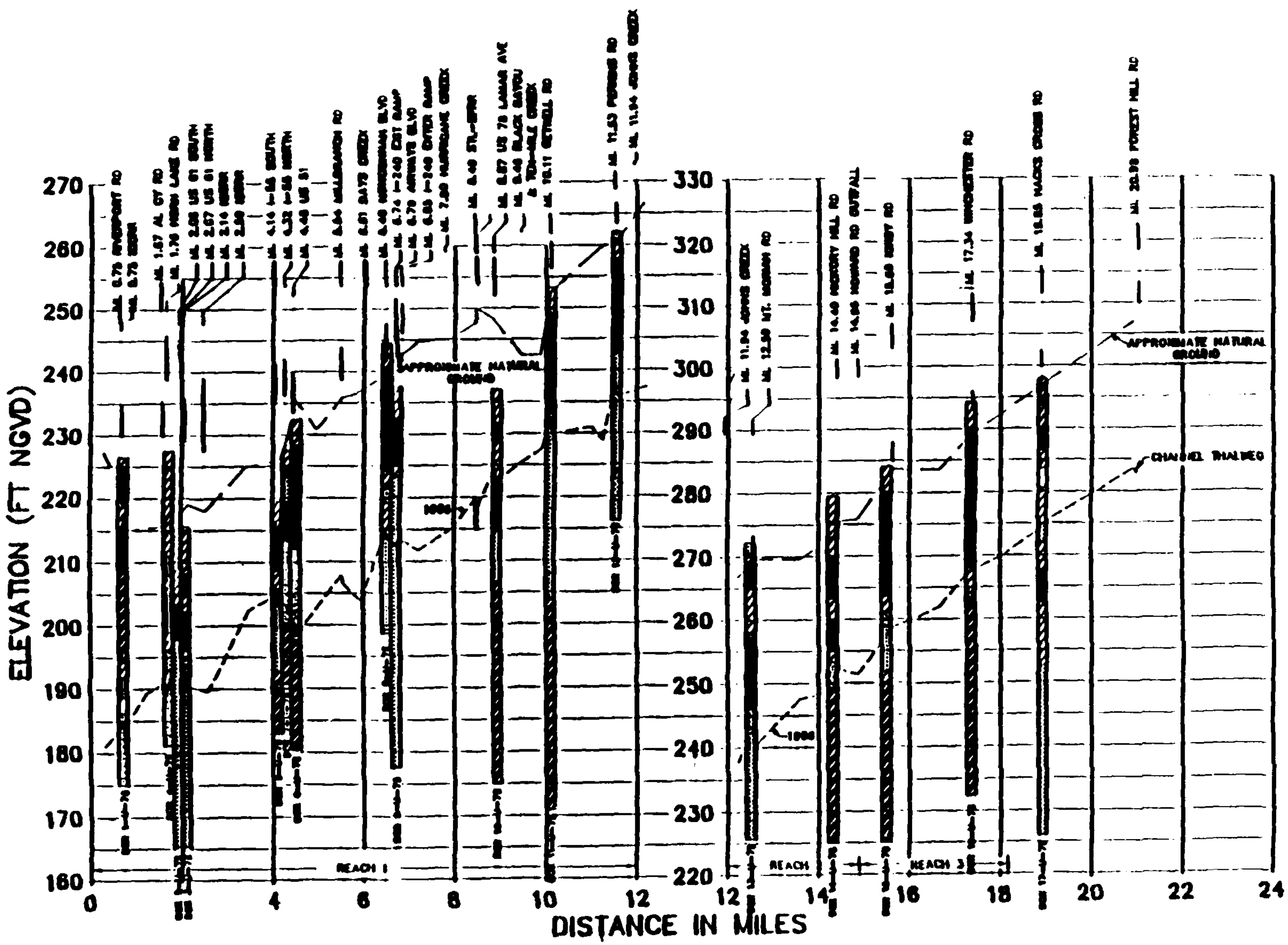


Figure 4. Soil characteristics

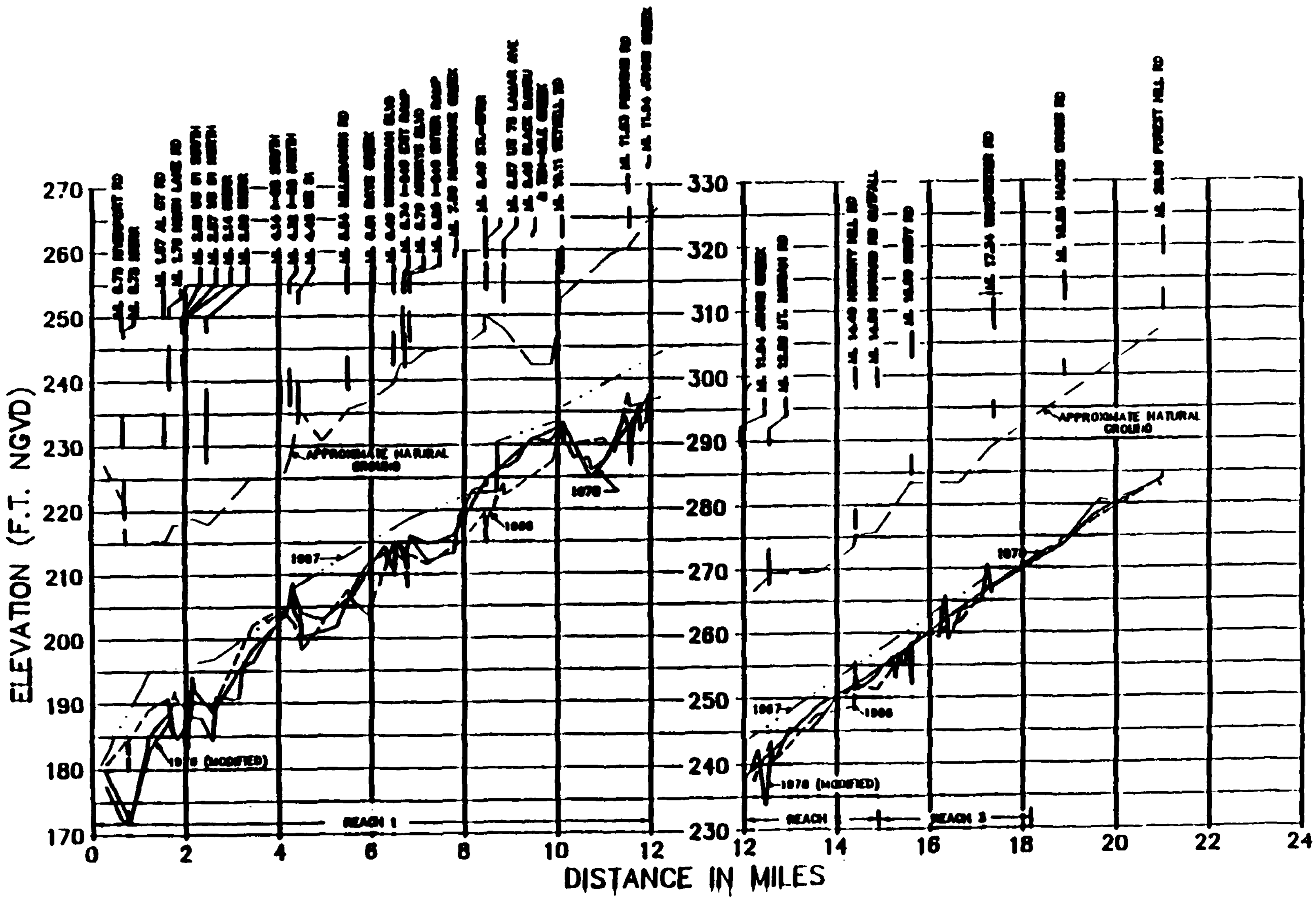


Figure 5. Historical bed profiles

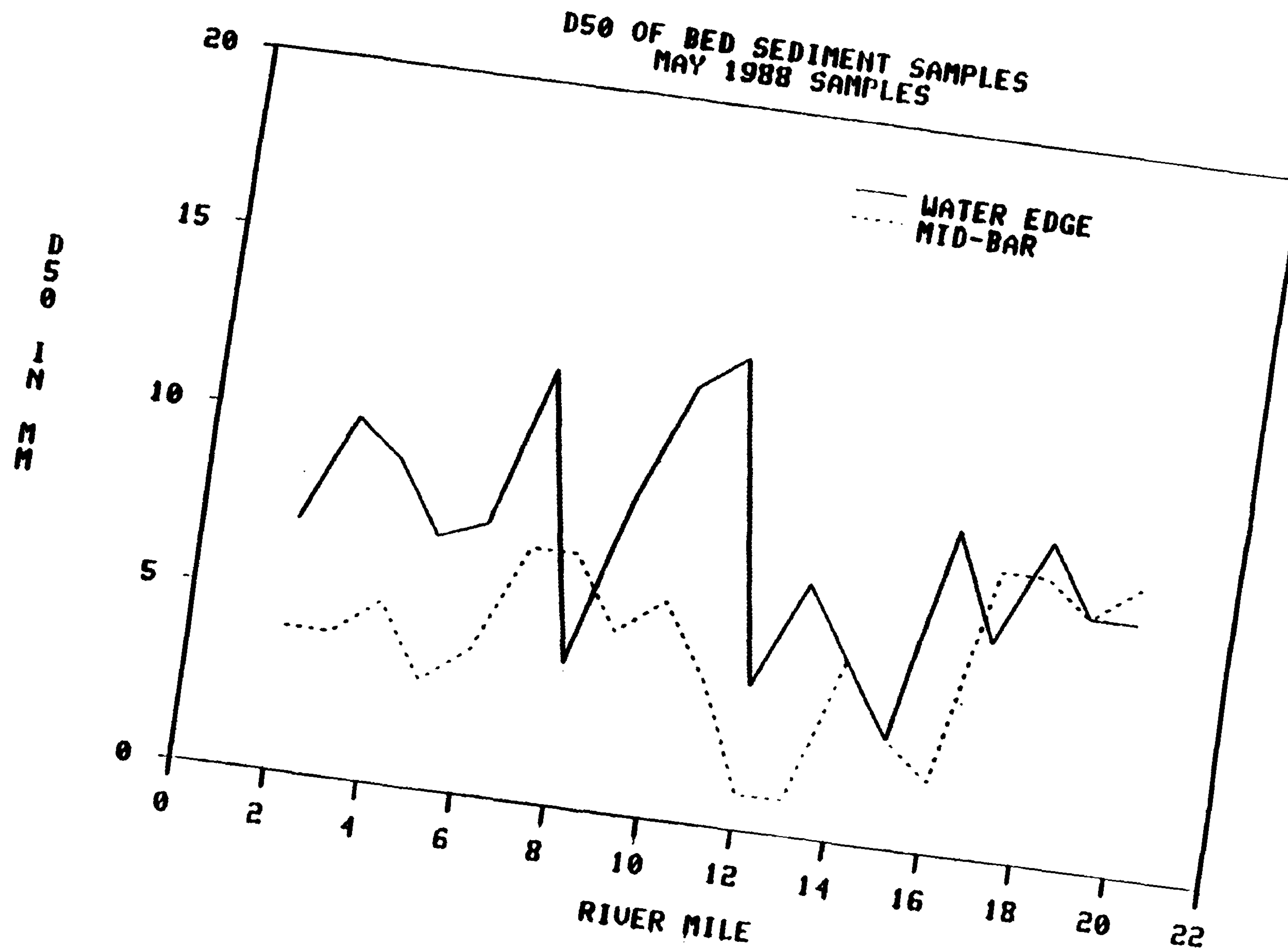


Figure 6. D50 of bed sediment samples

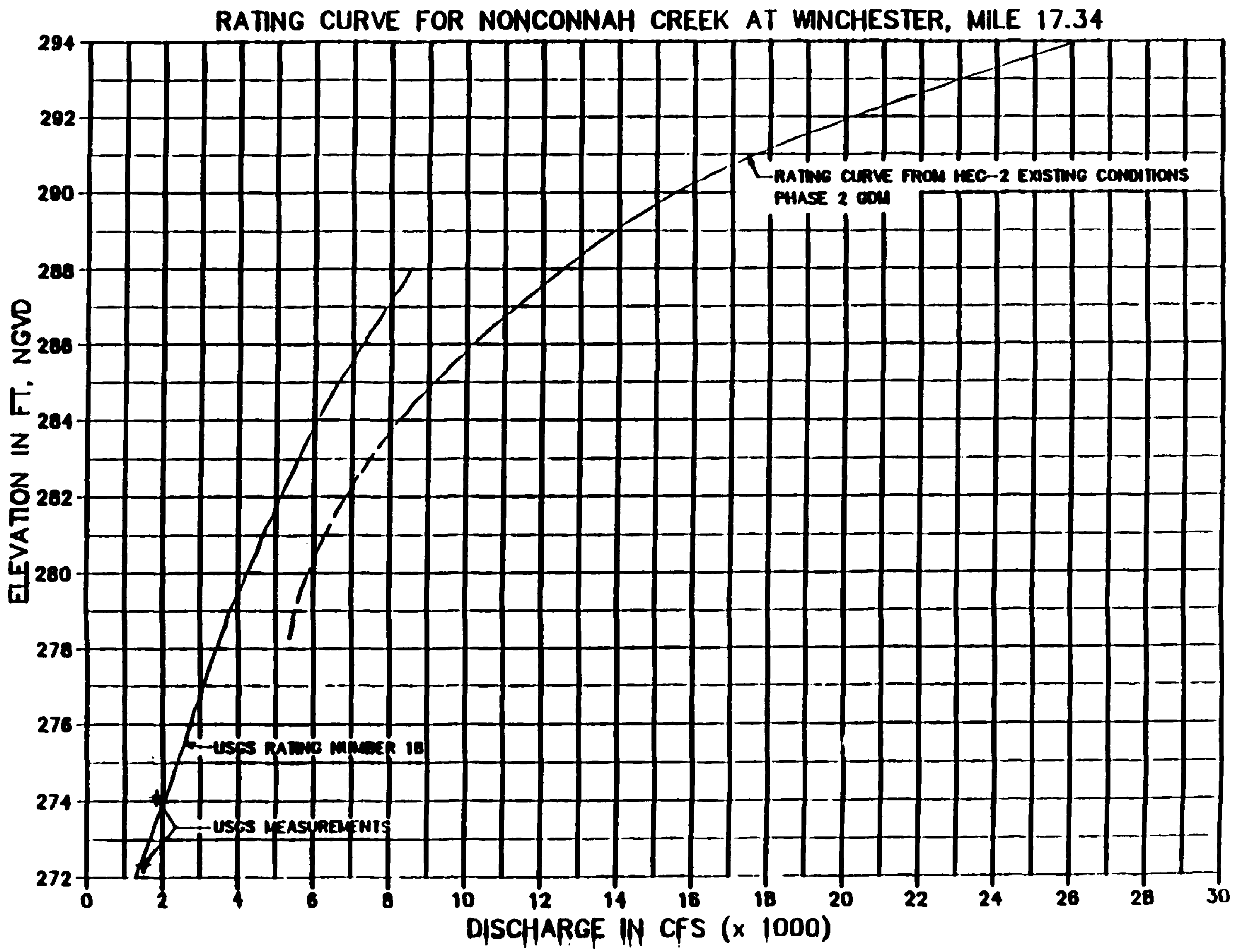


Figure 7. Rating curve at USGS gage, Winchester Road (RM 17.34)

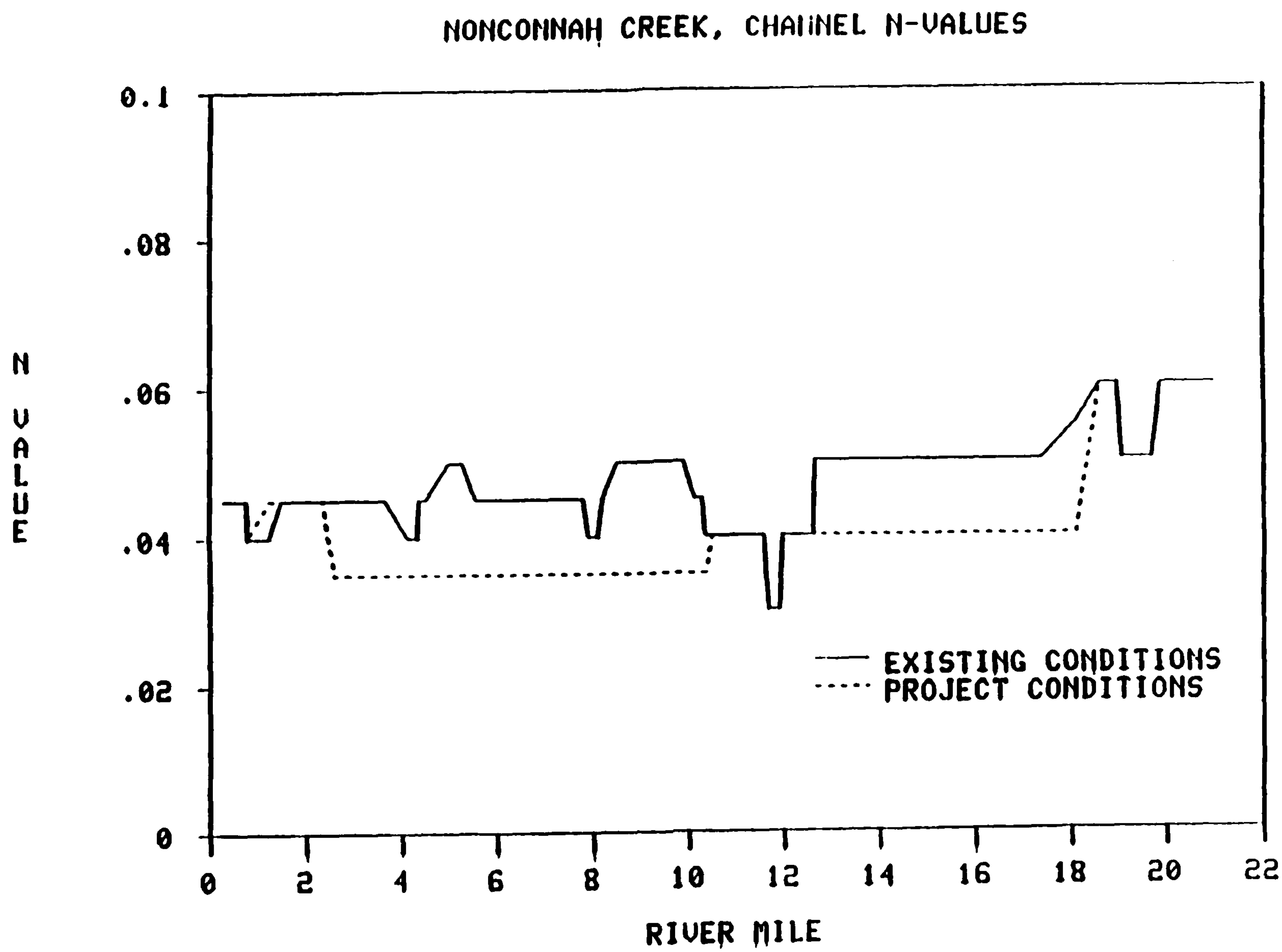


Figure 8. Nonconnah Creek, channel n values

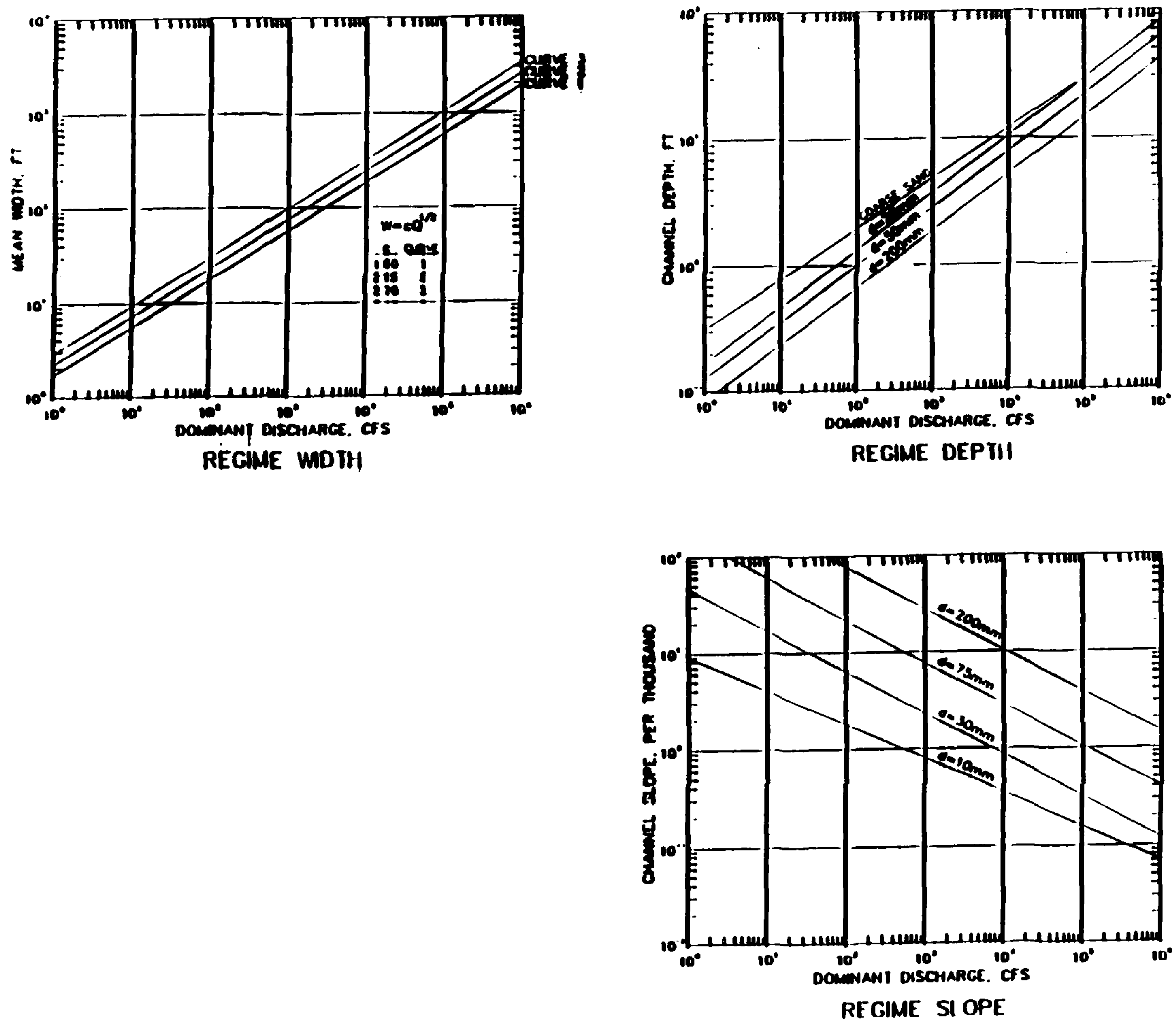


Figure 9. Regime relations of width, depth and slope

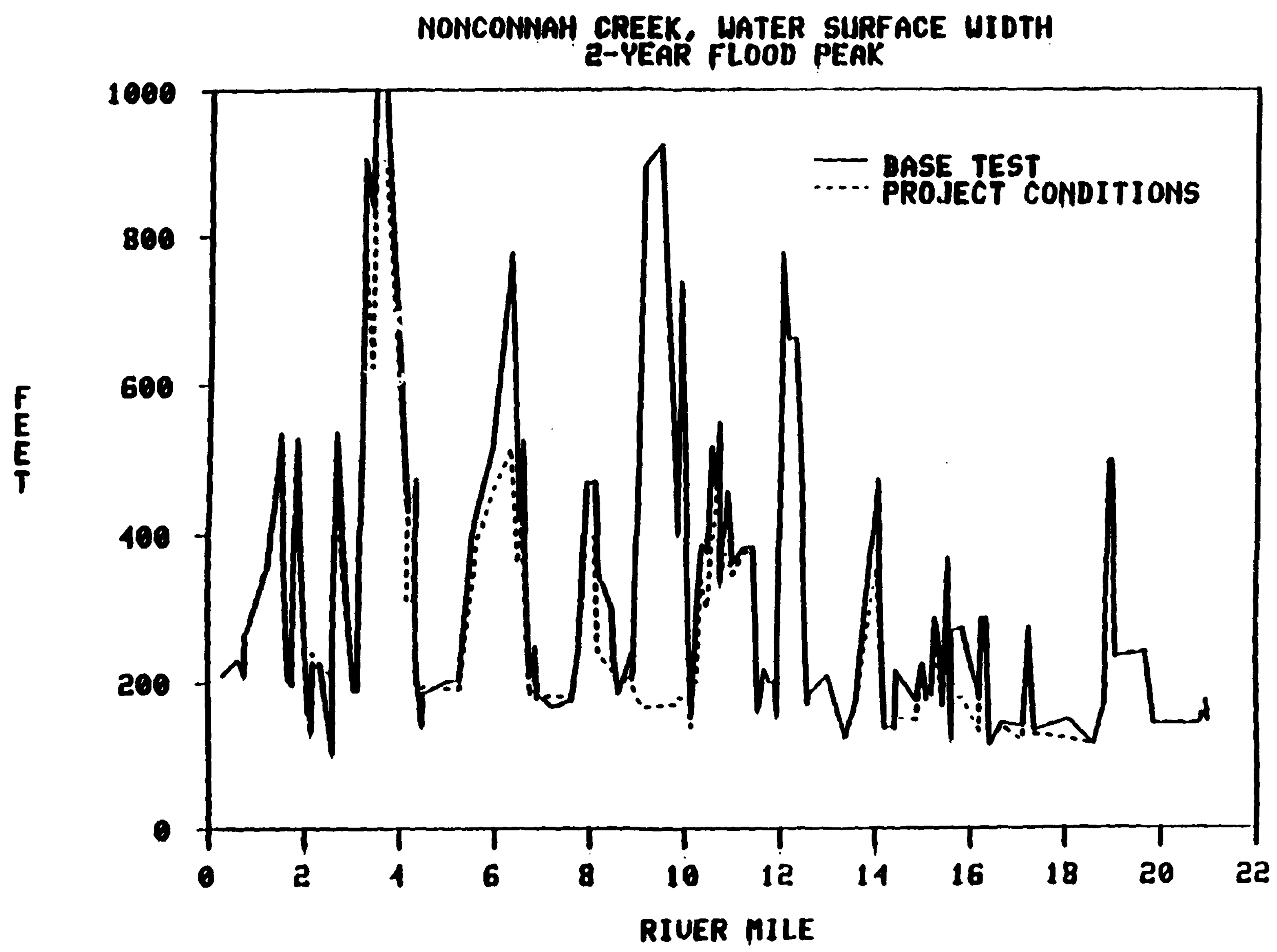


Figure 10. Nonconnah Creek, calculated water-surface width

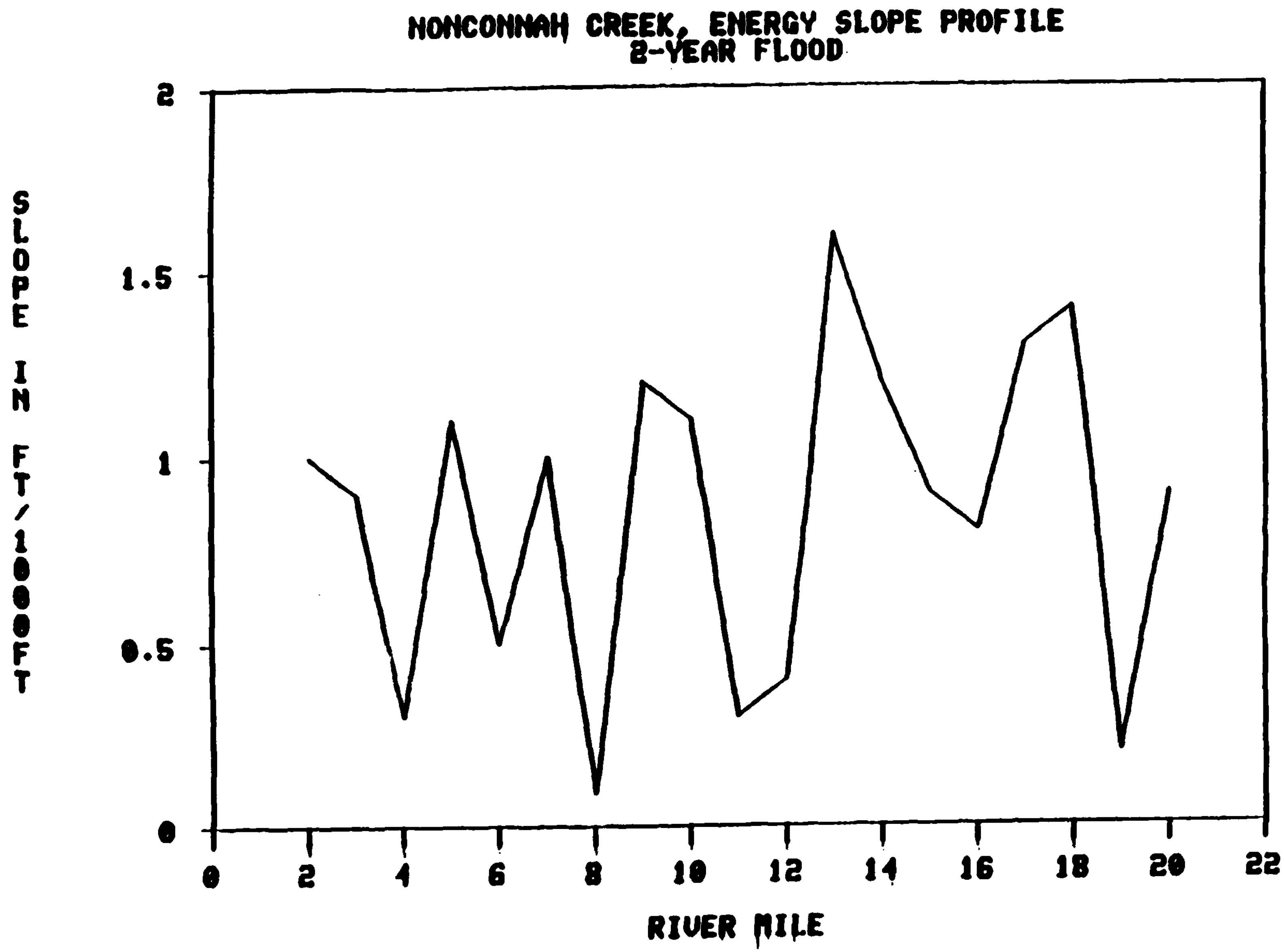


Figure 11. Calculated energy slope profile, existing conditions

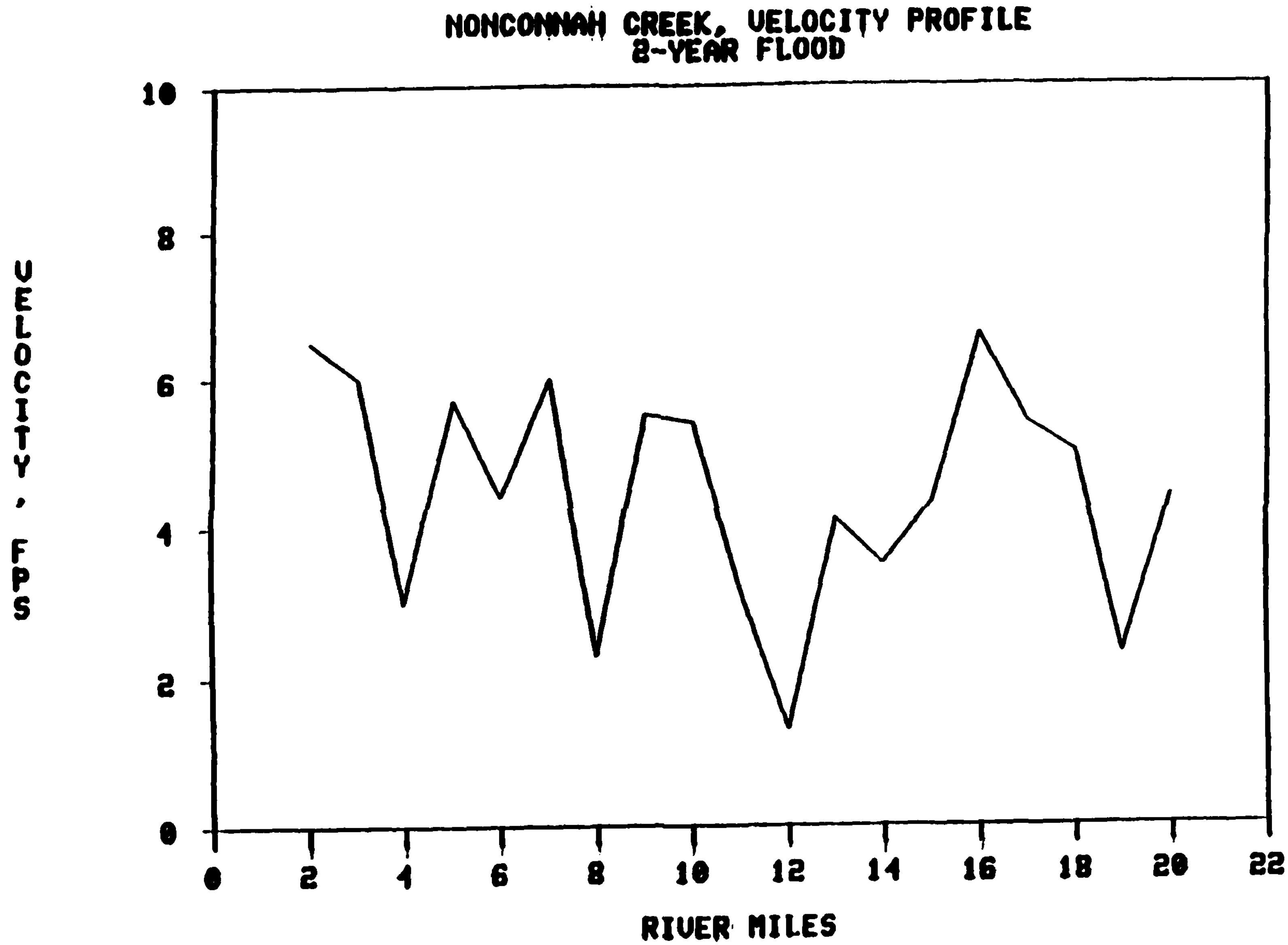


Figure 12. Calculated velocity profile, existing conditions

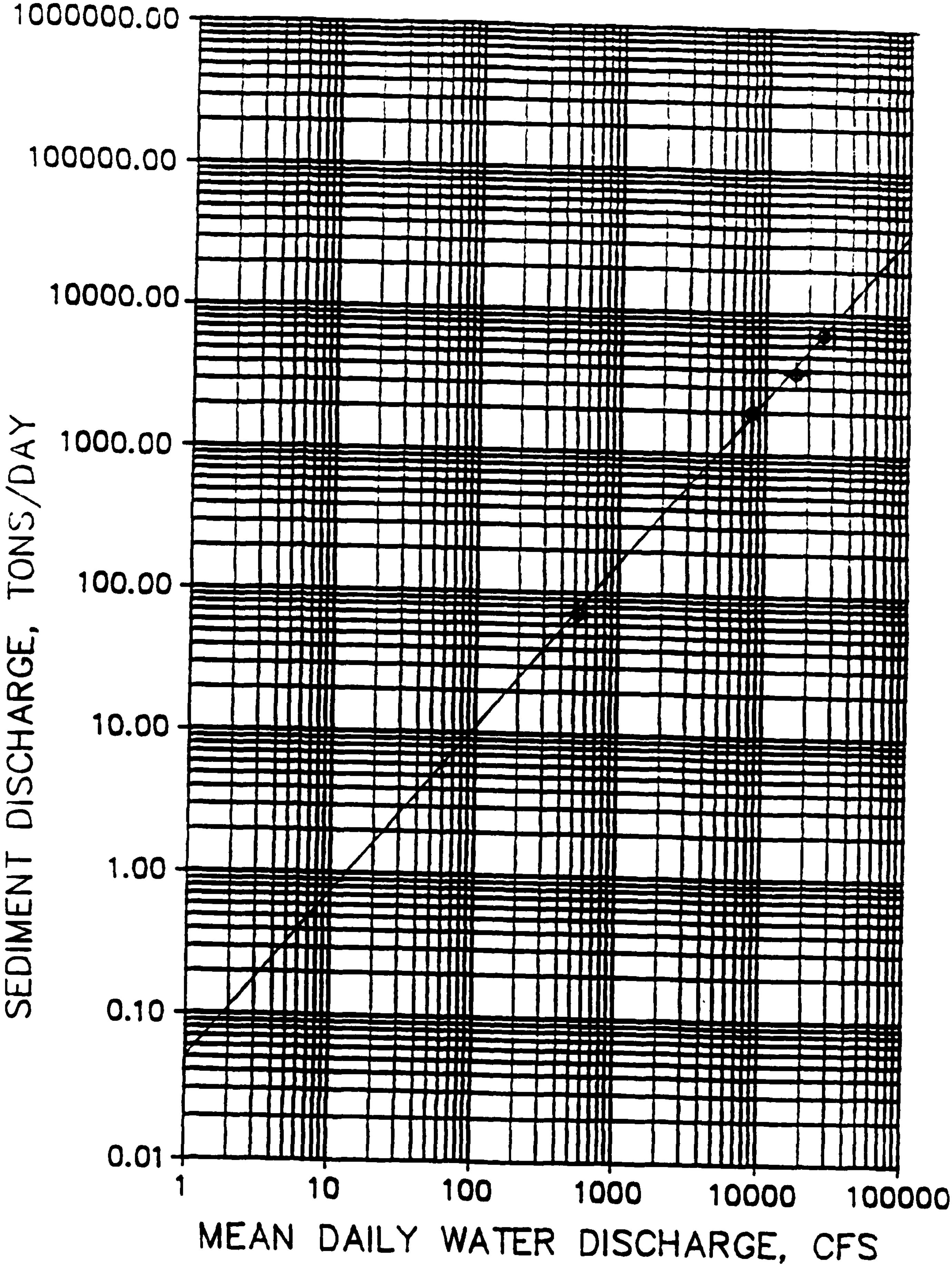


Figure 13. Calculated sediment inflow to project reach

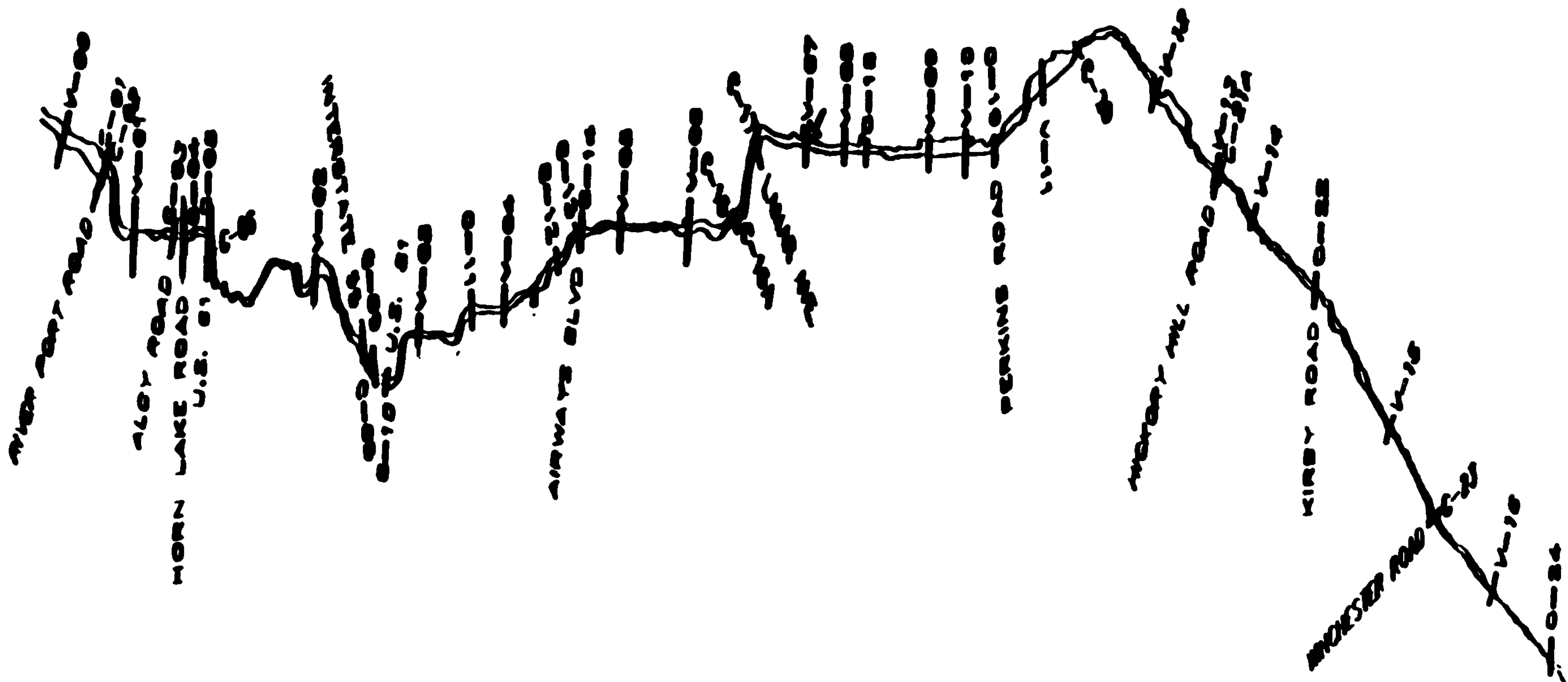


Figure 14. Cross section locations

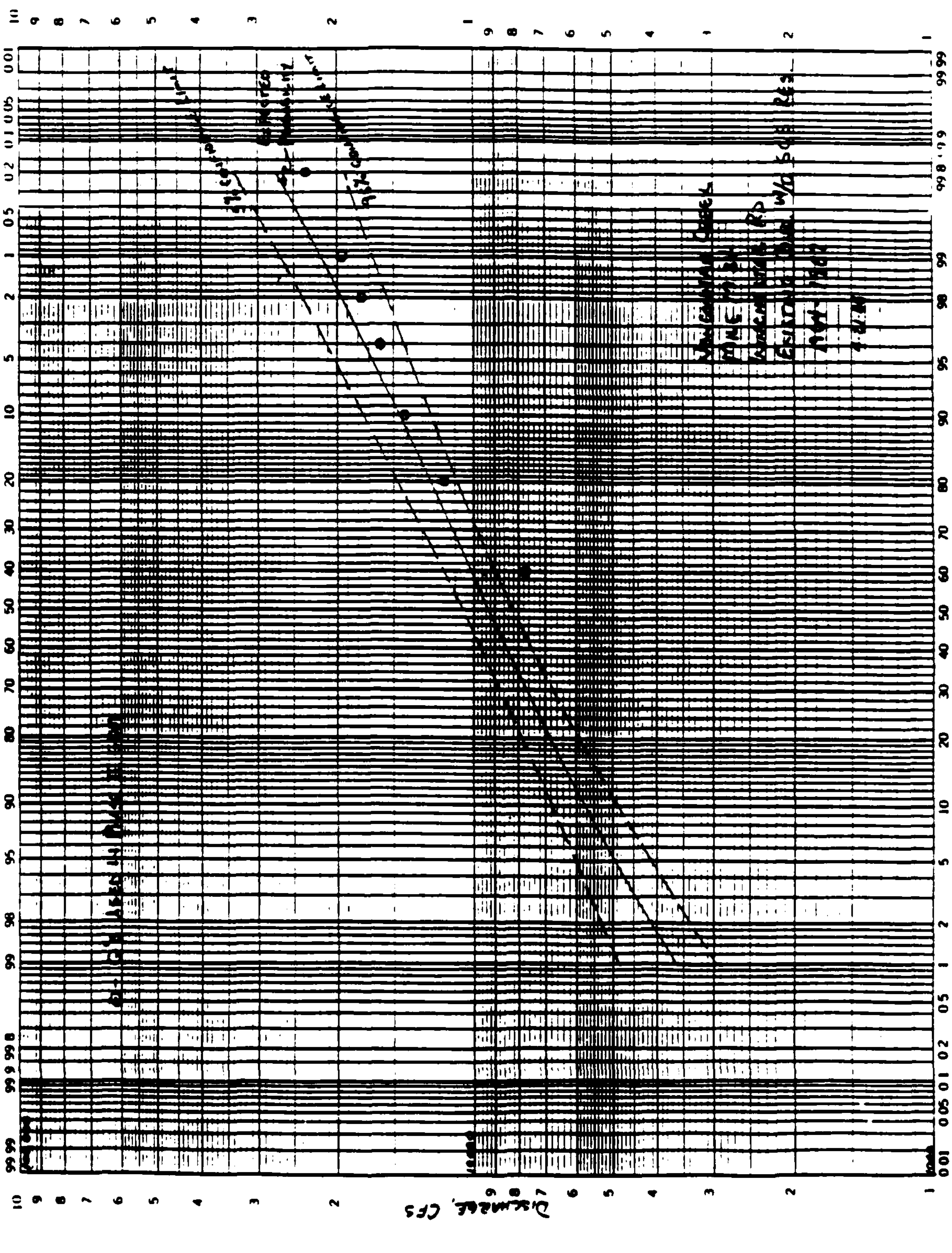


Figure 15. Annual peak discharge frequency, existing conditions

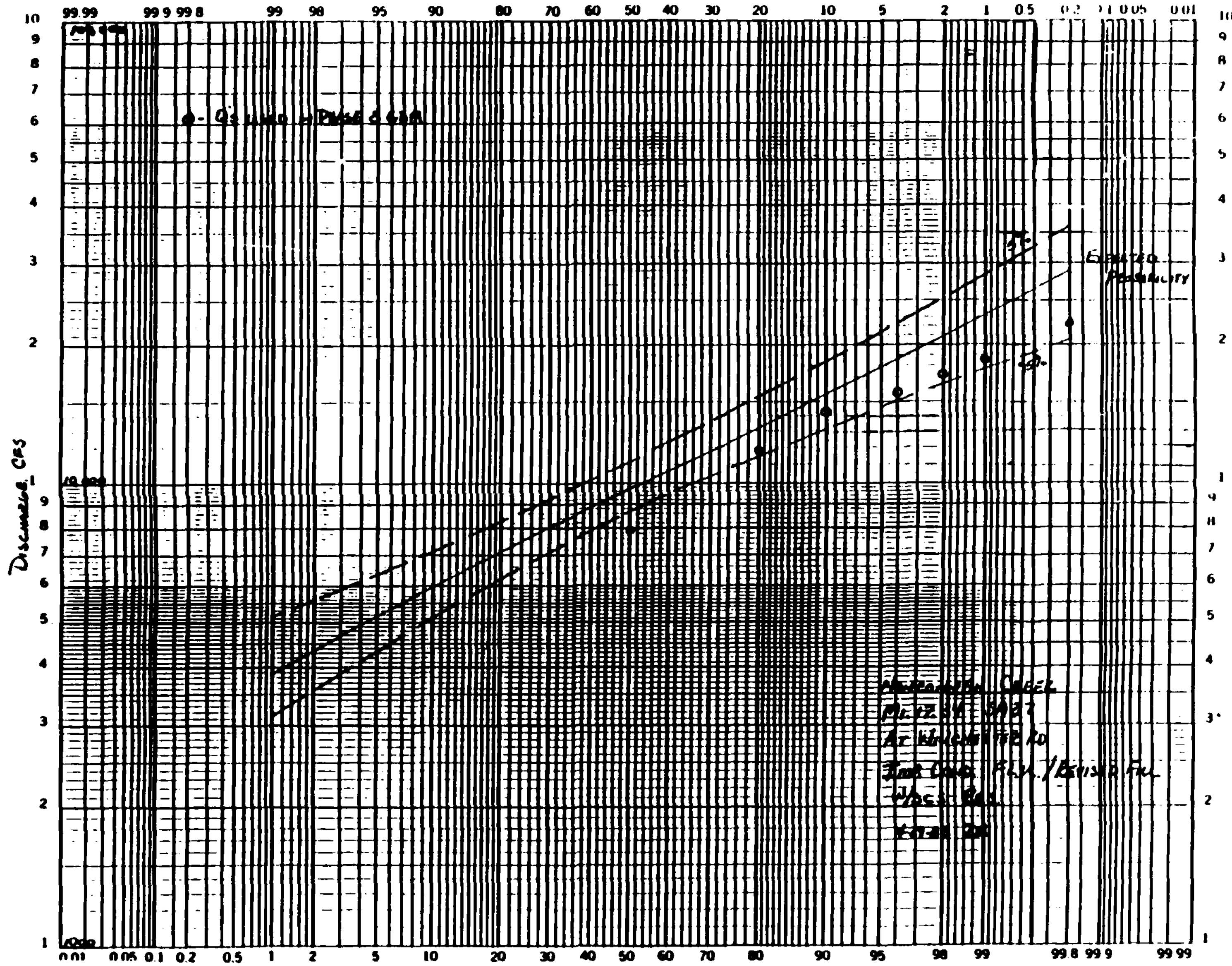


Figure 16. Annual peak discharge frequency, future conditions

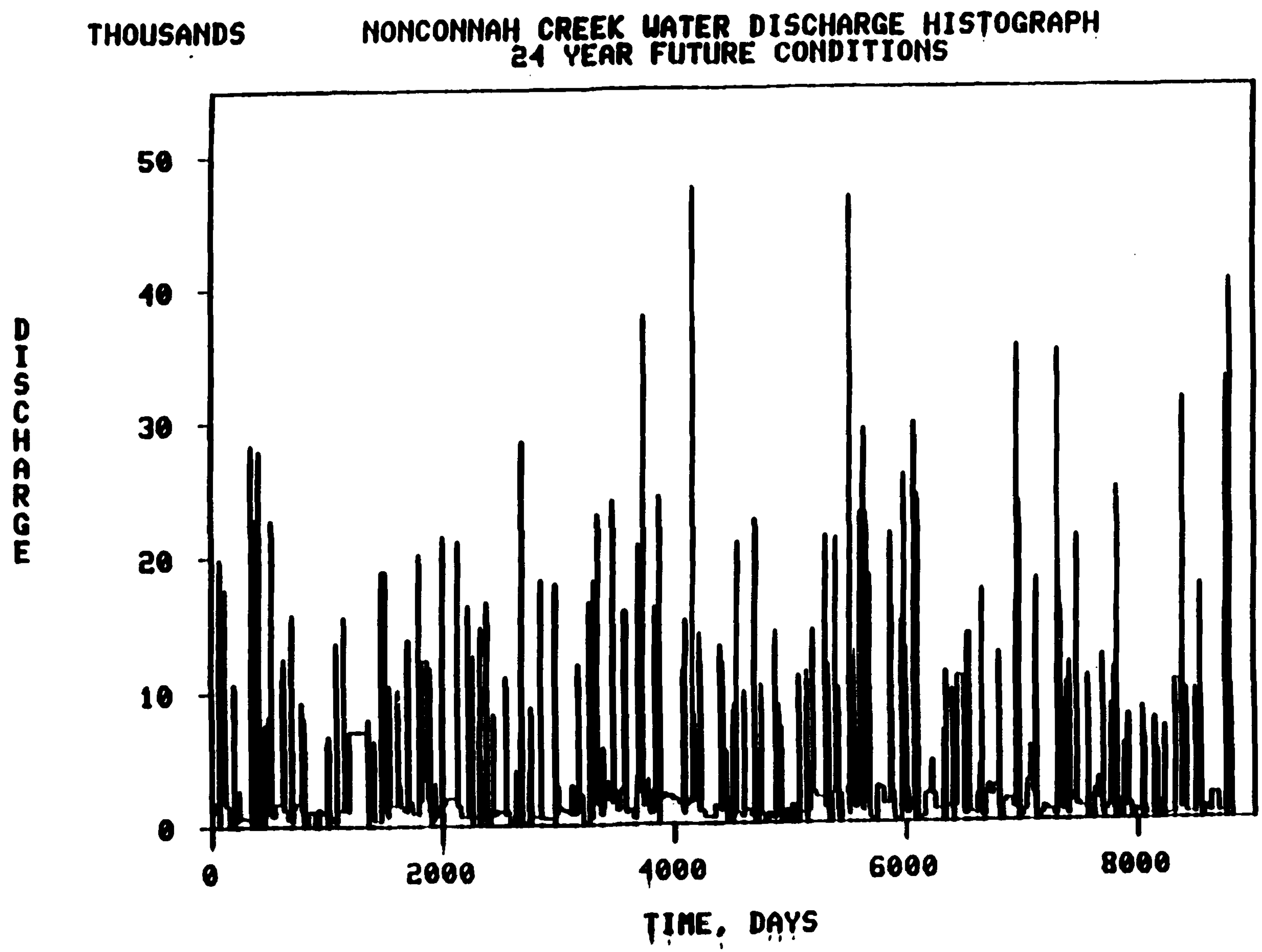


Figure 17. Water discharge histograph, 1964-1988

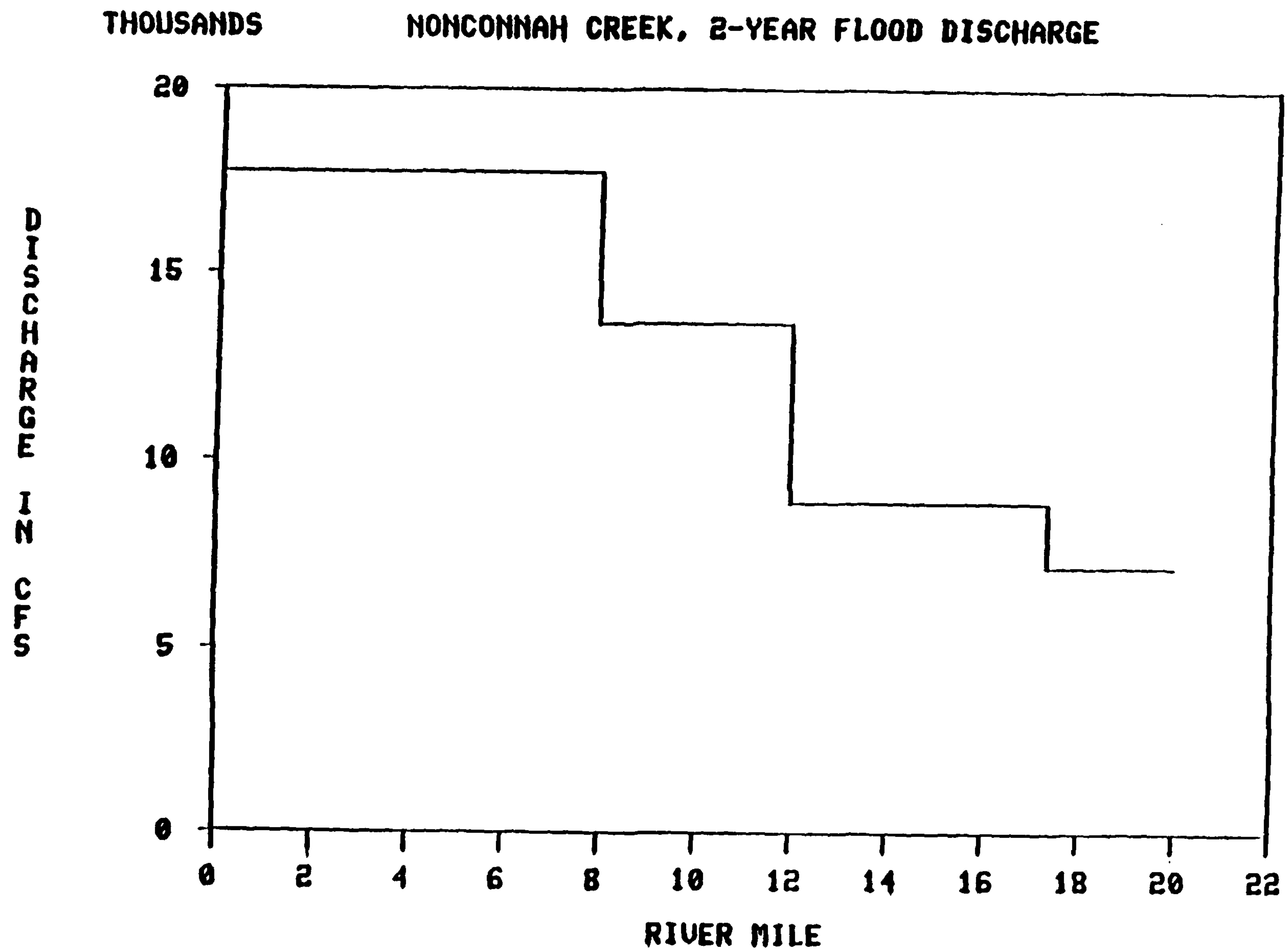
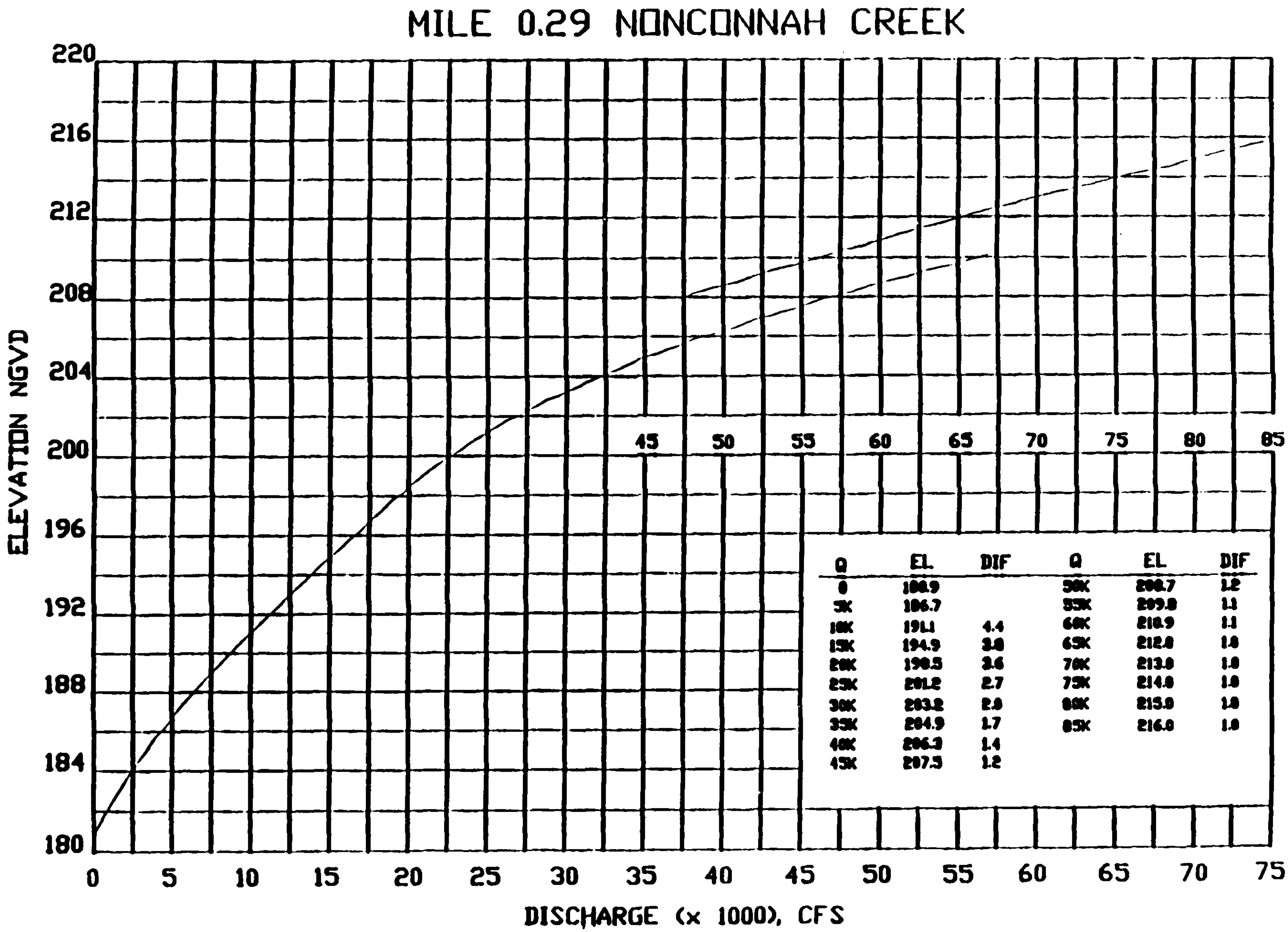


Figure 18. 2-Year flood discharge



Q	EL	DIF	Q	EL	DIF
0	180.9		50K	208.7	1.2
5K	186.7		55K	209.8	1.1
10K	191.1	4.4	60K	210.9	1.1
15K	194.9	3.8	65K	212.0	1.0
20K	198.5	3.6	70K	213.0	1.0
25K	201.2	2.7	75K	214.0	1.0
30K	203.2	2.0	80K	215.0	1.0
35K	204.9	1.7	85K	216.0	1.0
40K	206.3	1.4			
45K	207.5	1.2			

Figure 19. Normal depth rating curve, RM 0.29.

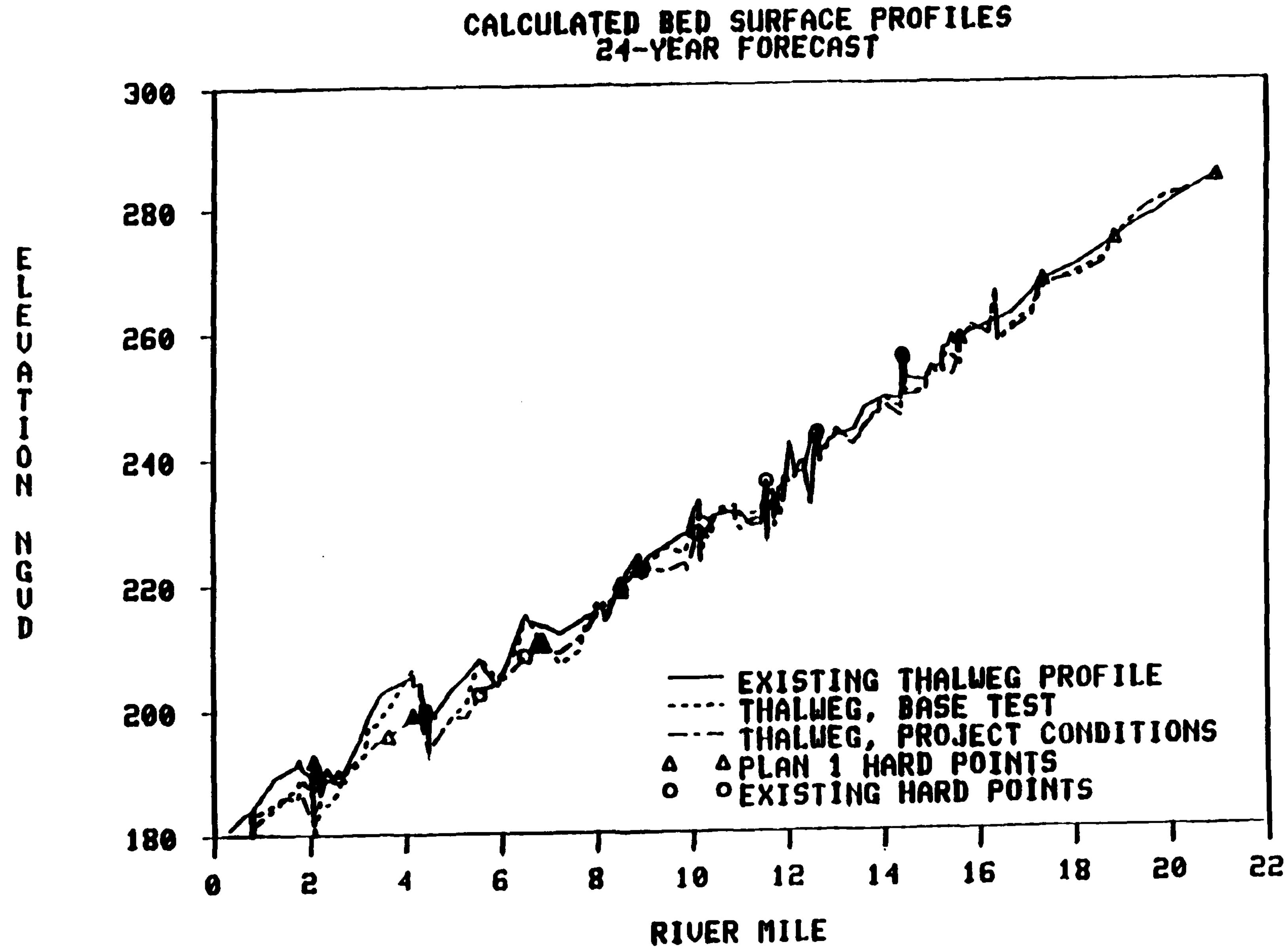


Figure 20. Calculated bed-surface profiles, 24-year forecast

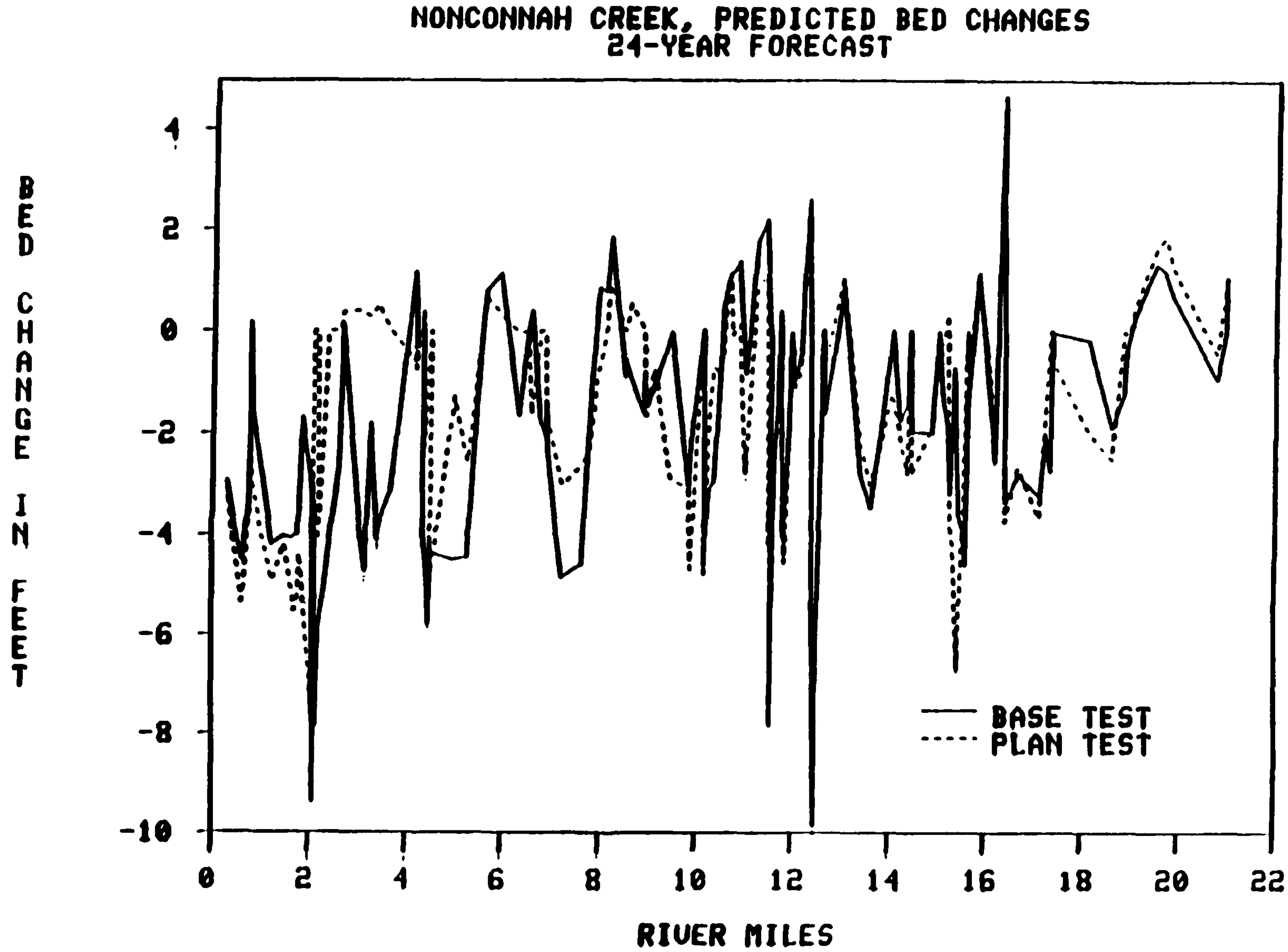


Figure 21. Predicted bed changes, 24-year forecast

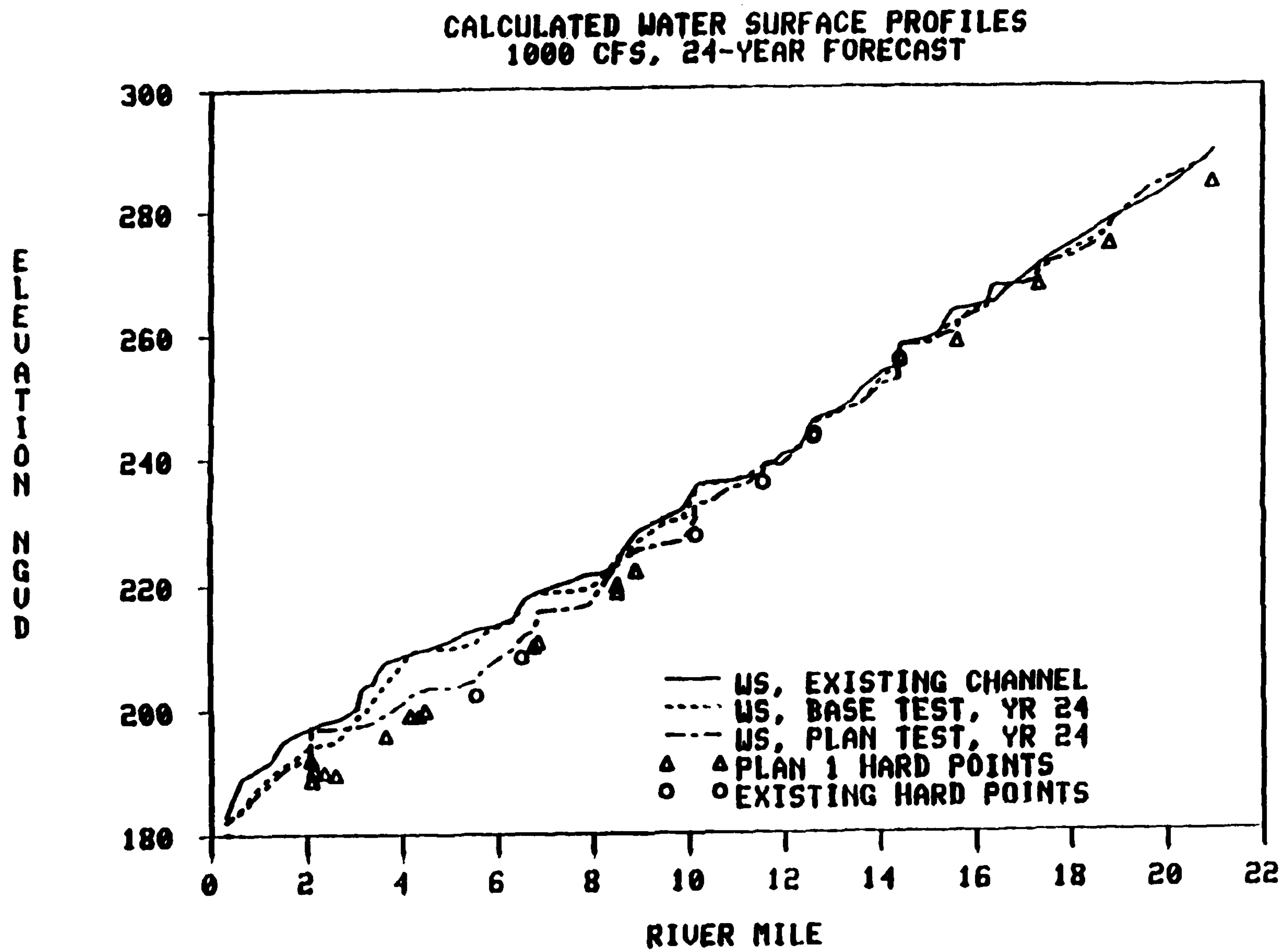


Figure 22. Calculated water-surface profiles, 24-year forecast

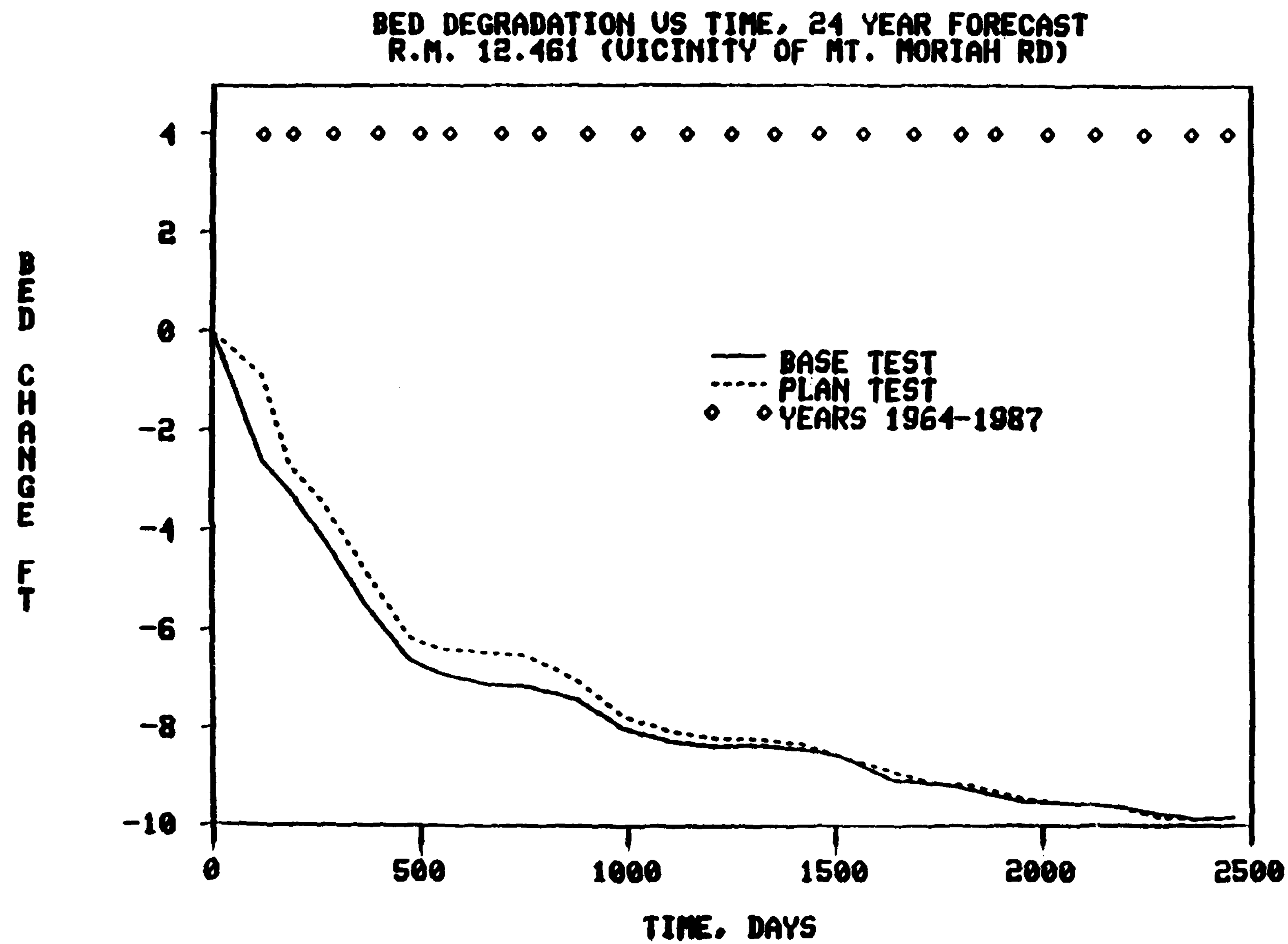


Figure 23. Bed degradation vs time, 24-year forecast, RM 12.461

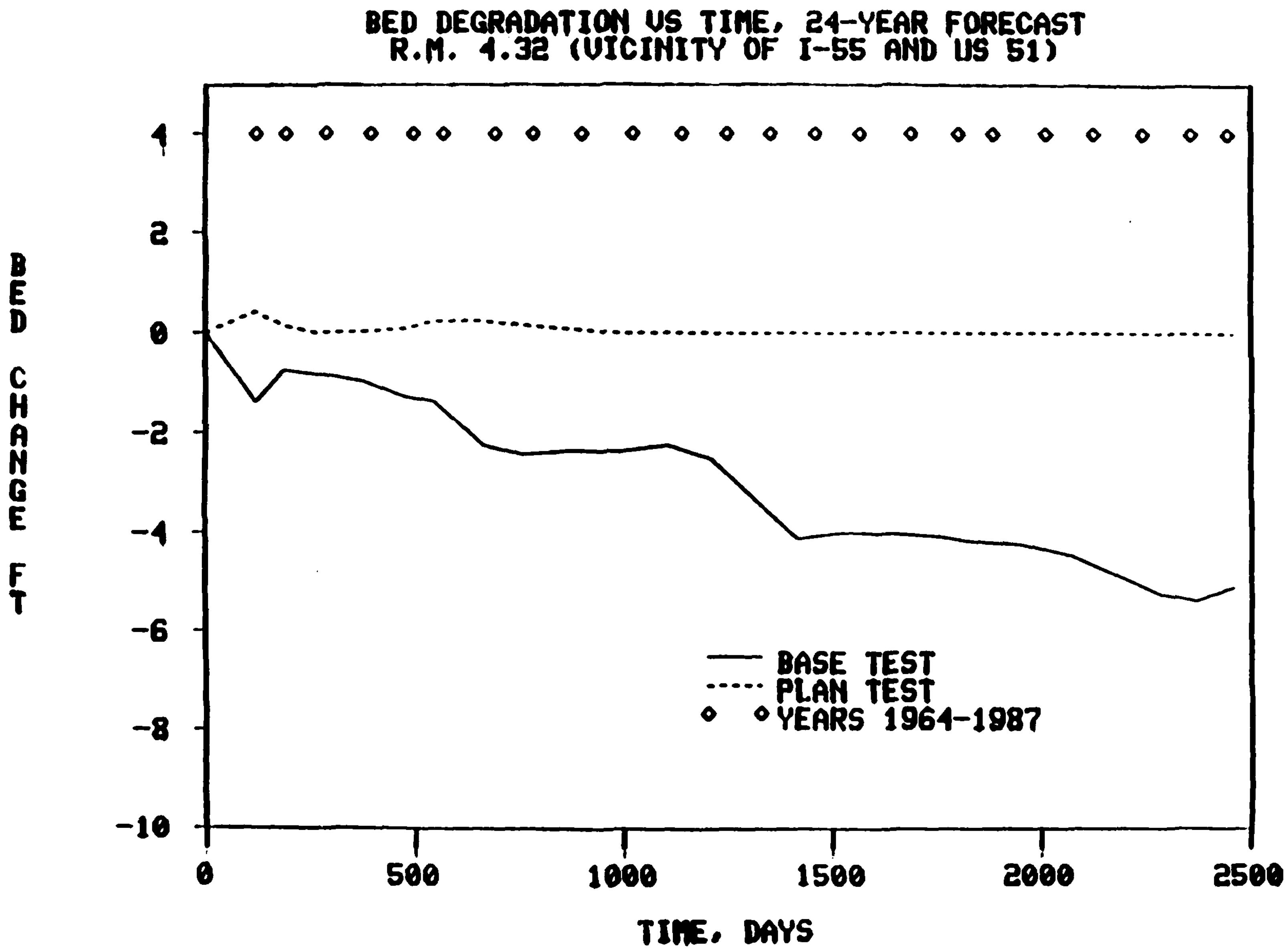


Figure 24. Bed degradation vs time, 24-year forecast, RM 4.32

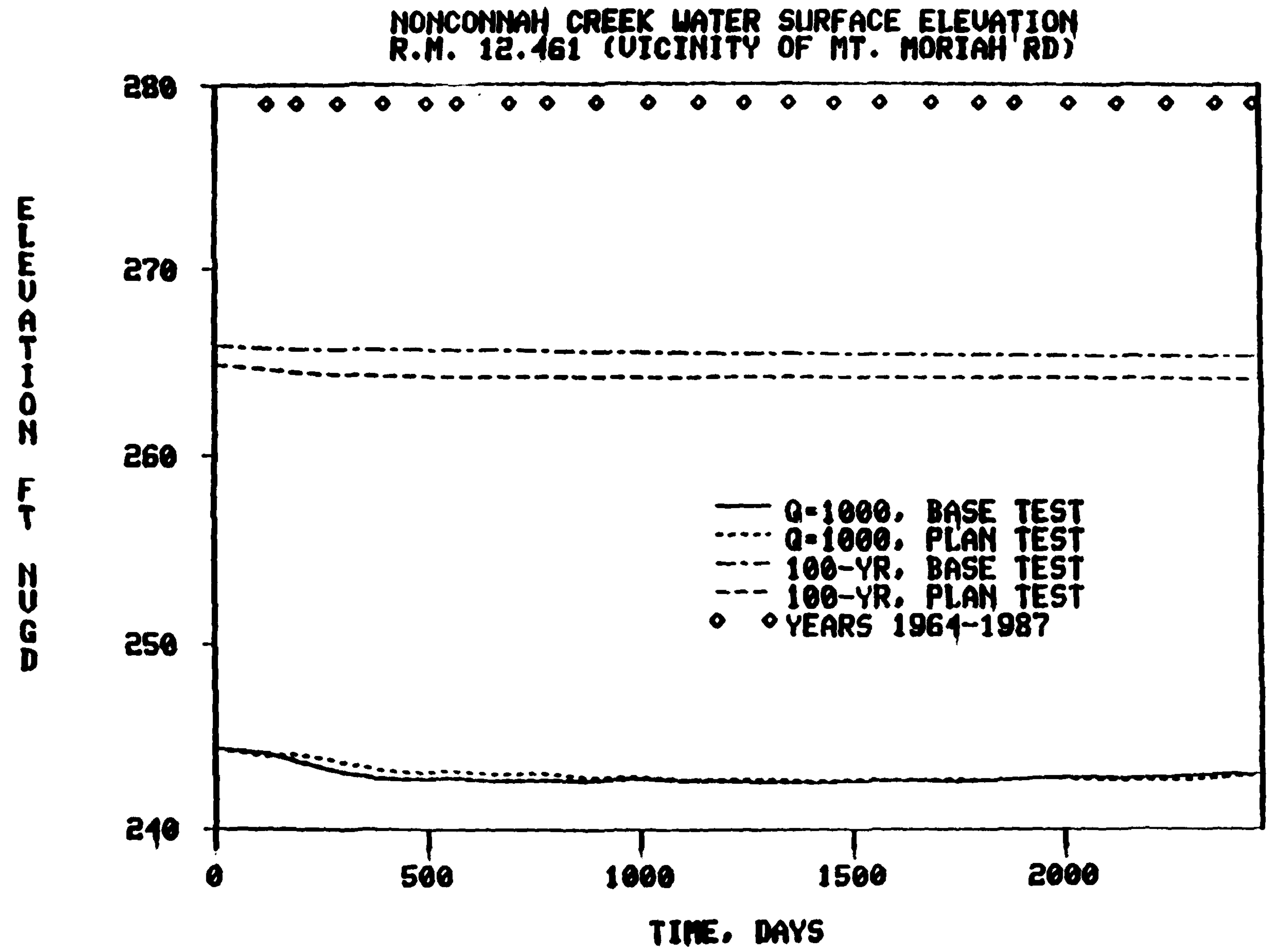


Figure 25. Water-surface elevation vs time, 24-year forecast, RM 12.461

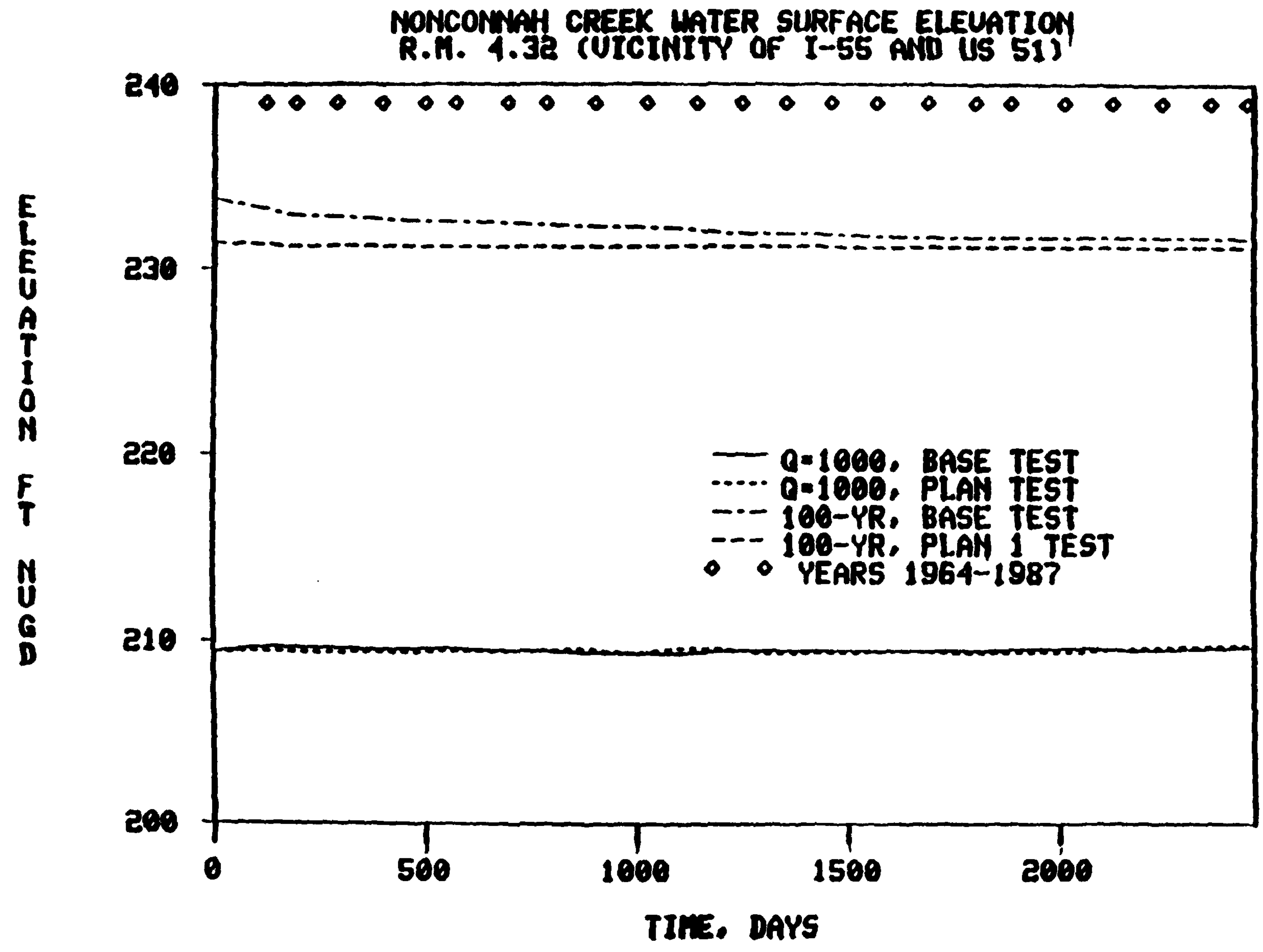


Figure 26. Water-surface elevation vs time, 24-year forecast, RM 4.32

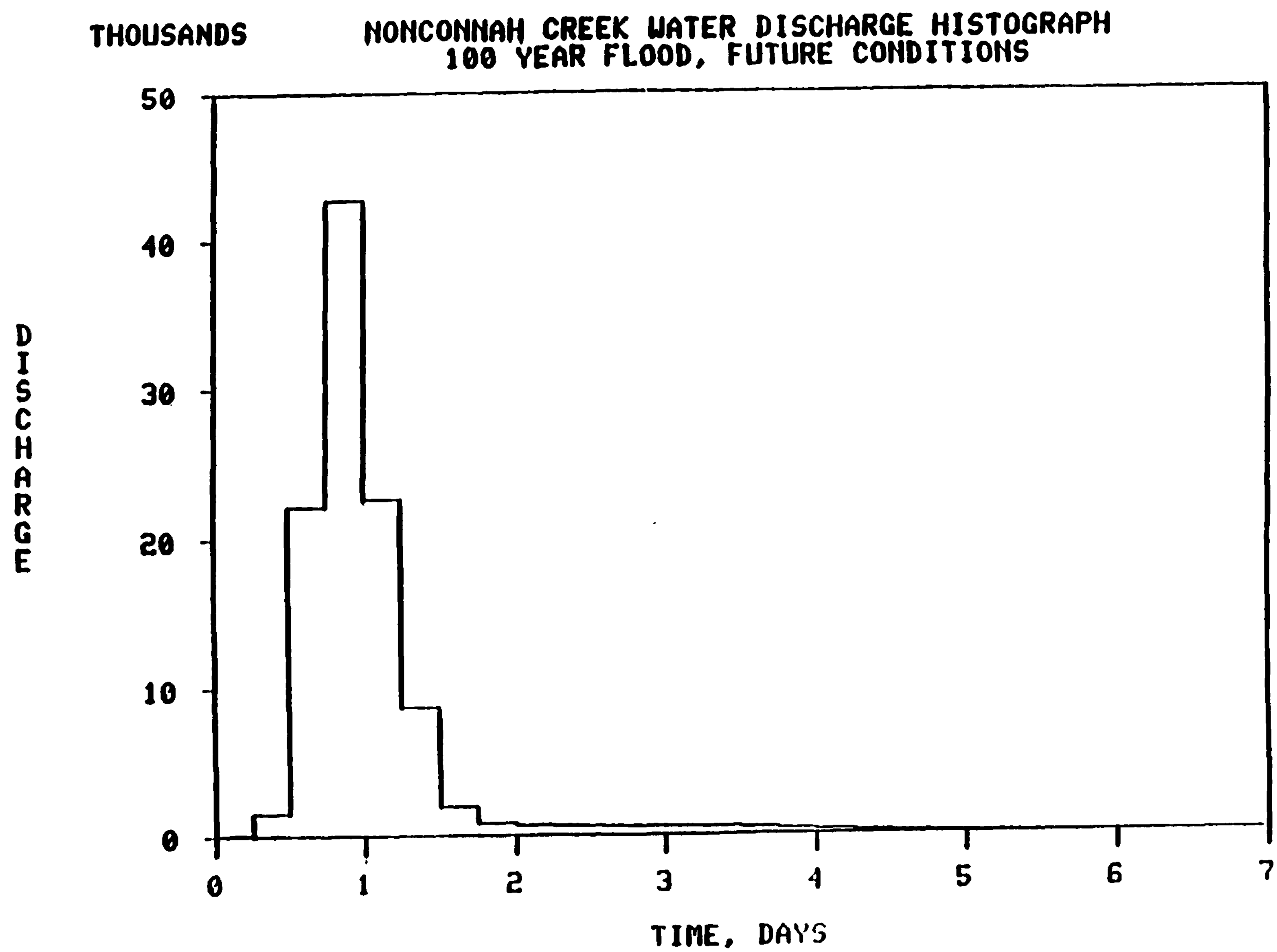


Figure 27. Water discharge histograph, 100 year flood

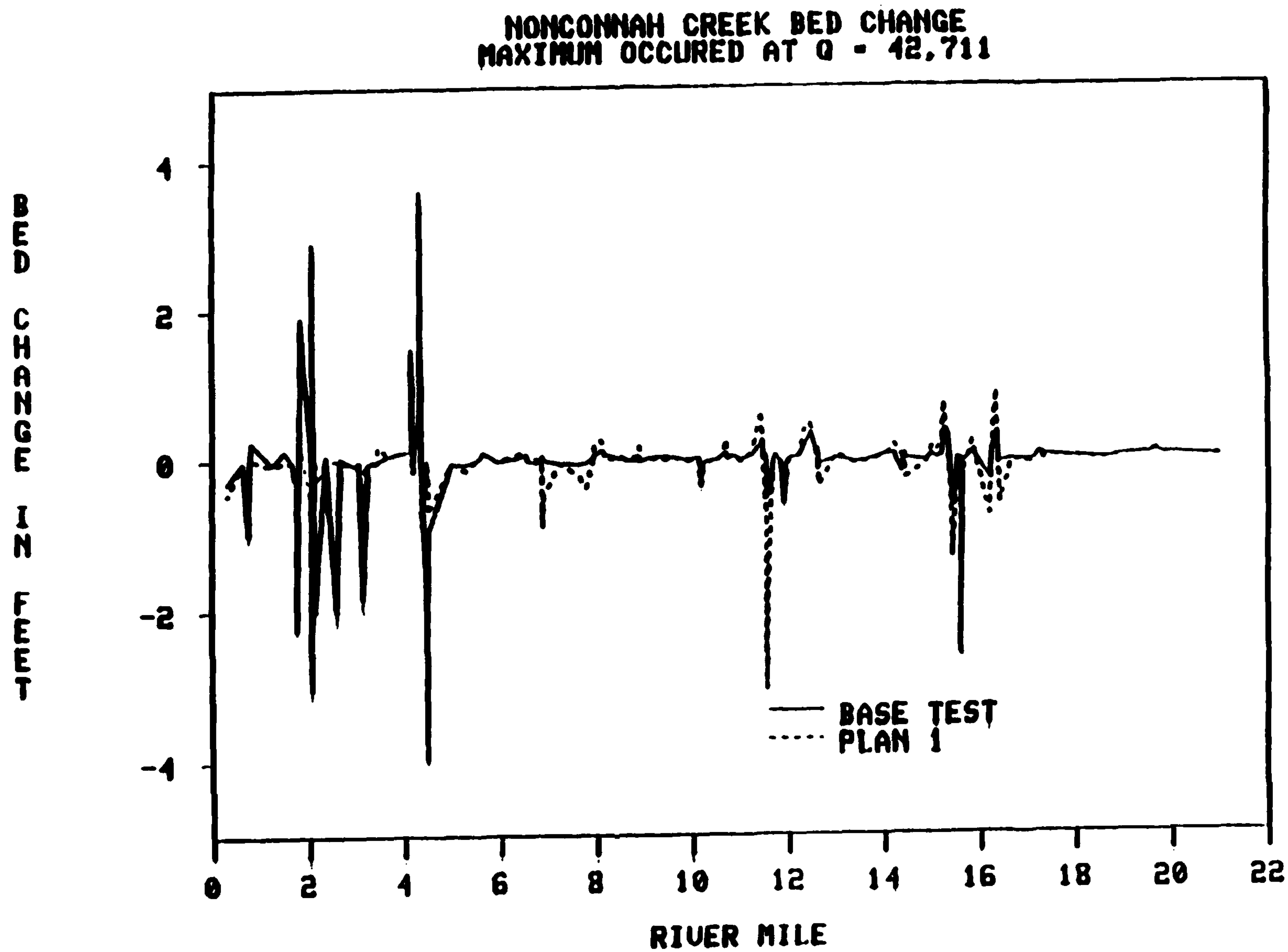


Figure 28. Calculated change in bed elevation at peak discharge

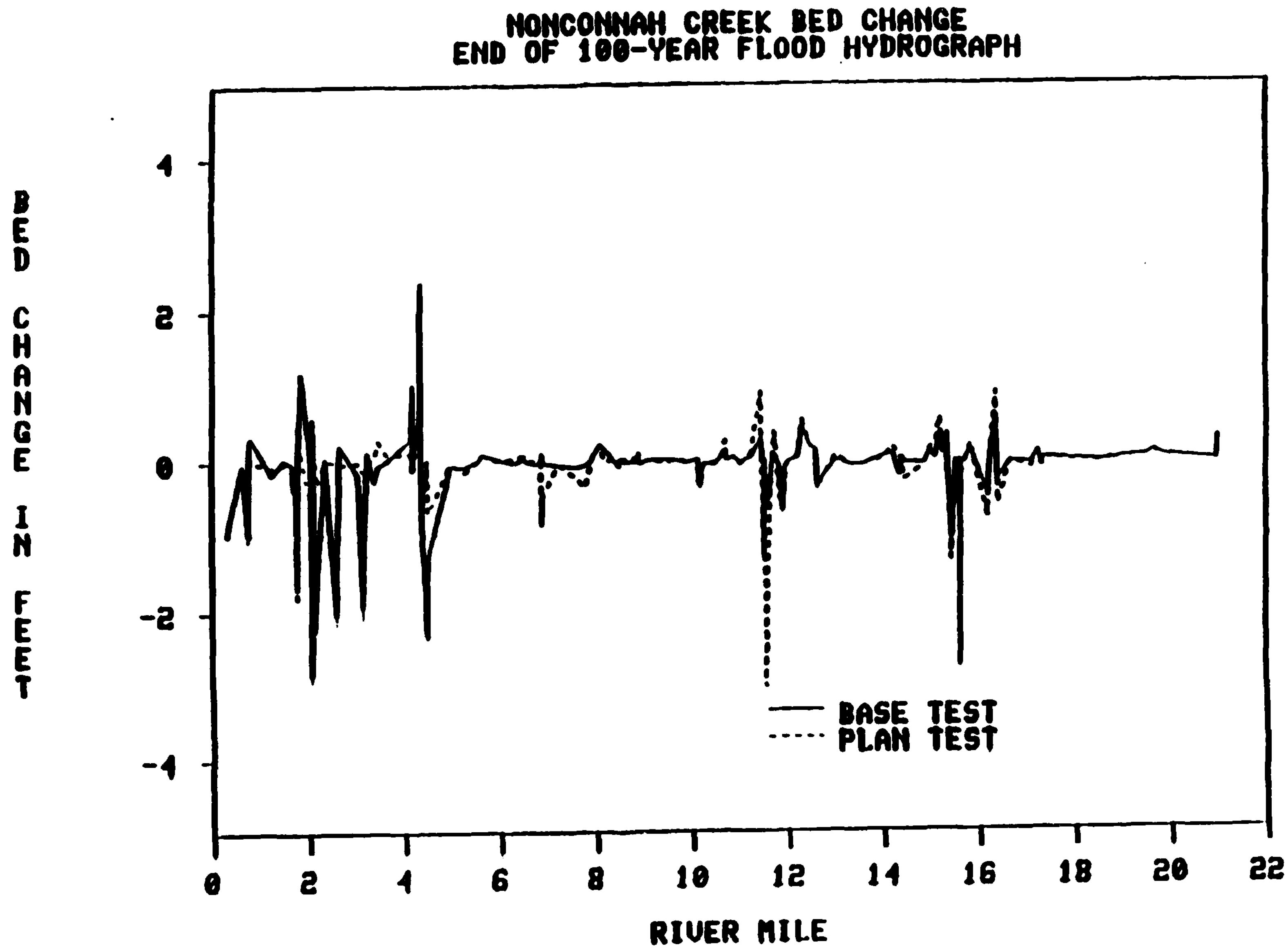


Figure 29. Calculated change in bed elevation, end of the 100-year flood

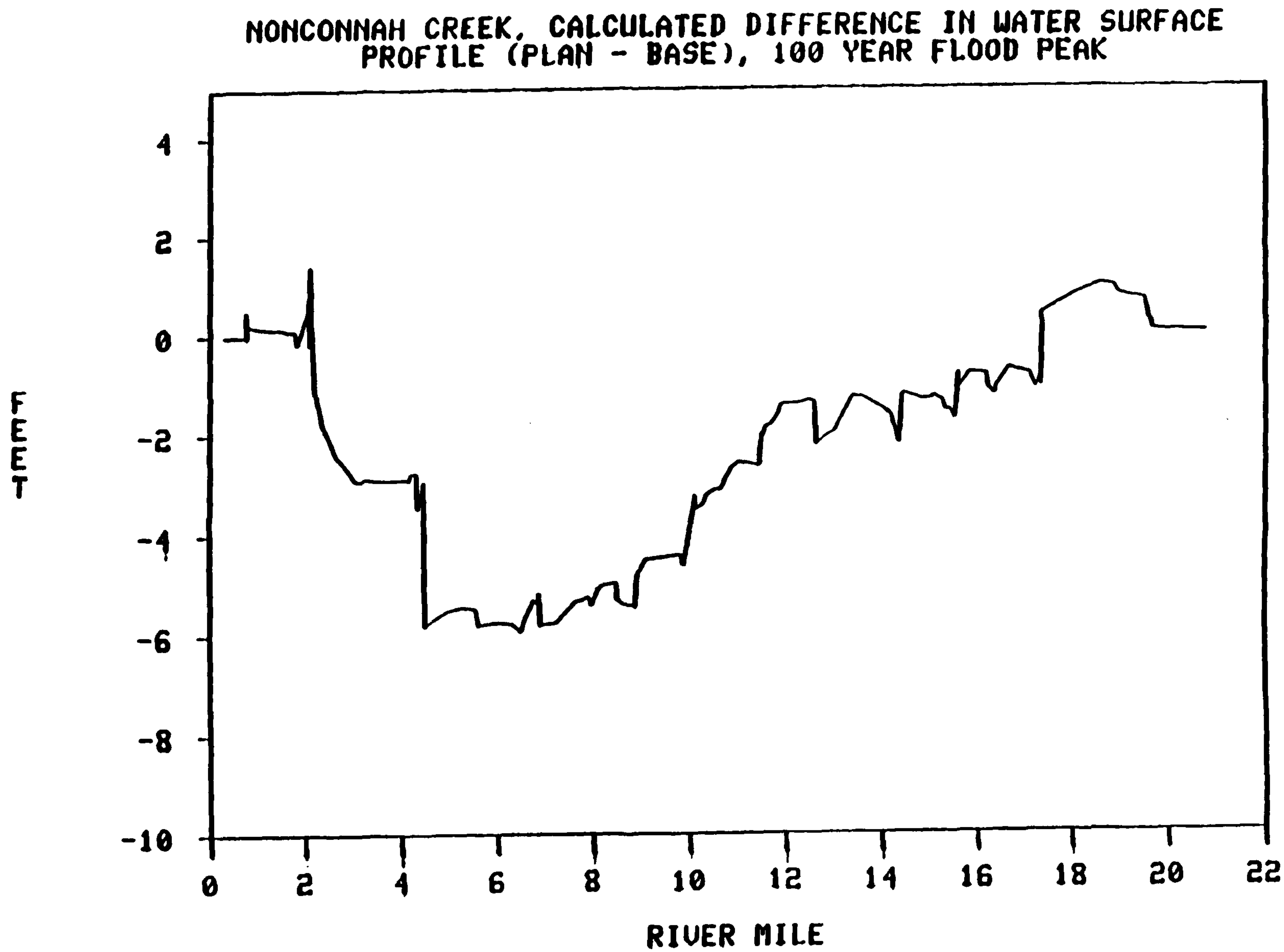


Figure 30. Calculated water-surface change with project, peak of the 100-year flood